TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOLUME 125, PART II
1960

Edited by the Executive Secretary, under the direction of the Committee on Publications.

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NEW YORK
PUBLISHED BY THE SOCIETY
1961

RG27.8 A512s
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TECHNOLOGY & SCIENCE
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FOREWORD

The Transactions of the American Society of Civil Engineers, Vol. 125, 1960, consists of two parts, of which this is Part II. Vol. 125, Part II is a Symposium on Rockfill Dams, the "core" of which is a series of papers presented before the Power Division of ASCE in June 1958 at Portland, Oregon. The decision to issue this second part to Vol. 125 was made by the ASCE Committee on Publications in June 1960 in Reno, Nevada, when it became apparent that the interest in this subject warranted the publication of a volume of Transactions devoted exclusively to the science of rockfill dams.

INTRODUCTION

SYMPOSIUM ON ROCKFILL DAMS

Planning for the Symposium on Rockfill Dams began in early 1957 and seventeen papers were presented at the June 1958 ASCE Convention in Portland, Oregon. To keep the coverage up to date and to expand it, discussions were arranged over the 2½ year period to the end of 1960. The ASCE Committee on Publications decided, in June 1960, that the Symposium was of substantial permanent value and should be published in a single Transactions volume. This volume has been arranged to include, with only two exceptions,[1,2] all ASCE literature on rockfill dams for the years 1954-1960.

For purposes of the Symposium, a rockfill dam was defined as one that relies on rock, either dumped in lifts or compacted in layers, as a major structural element. Included are rockfill dams with (1) impervious face membranes, (2) sloping earth cores, (3) thin central cores, and (4) thick central cores. The papers were planned to provide design, construction, and performance data on most of the world's higher rockfill dams. The discussions were arranged to introduce data on additional dams and to bring the performance data up to date on the dams in the original papers. The detailed presentation of settlement data was emphasized as being of particular value in the field of rockfill dams.

Rockfill dams had their origin in the gold-mining regions of the Sierra Nevada in California, in the 1850's. Their development was logical; there was a need for dams in inaccessible locations where rock was abundant and the miners were well acquainted with the use of explosives. In comparison with the history of other types of dams, the history of rockfill dams is relatively short. Prior to 1925, there existed only eight rockfill dams higher then 100-ft, all of the impervious-face type. Through the 1930's a number of concrete face dams in the 200-ft to 300-ft range in height were constructed, the highest being the 328-ft high Salt Spring Dam completed in 1931. The 1940's saw the continued use of concrete face dams and the beginning of the large earth-core rockfill dams: The sloping core type, the 260-ft high Nantahala Dam, in 1942; the thick central core type, the 318-ft high Watauga Dam, in 1948; and the thin central core type, the 400-ft high Mud Mountain

---

[1] "Earthquake Resistance of Rockfill Dams," by Ray W. Clough and David Pirtz, Transactions, ASCE, Vol. 123, 1958, pp. 792-810.

[2] "Ambuklao Rockfill Dam, Design and Construction," by E. Montford Fucik and Robert F. Edbrooke, Transactions, ASCE, Vol. 125, Part I, 1960, pp. 1207-1227.

Dam, in 1948. In the period 1950-1960, much development occurred in the technology of rockfill dams and they were being constructed to ever-increasing heights that in 1960 are in the range of 400 ft. to 450 ft. The papers of the Symposium are arranged chronologically in this volume, by completion date of the dams, to present this history and development of the rockfill type of dam.

The early history of rockfill dams and the status of the technology in 1939 is well presented in a paper by J. D. Galloway and in the discussions by 25 engineers.[3]

Rockfill dams are being increasingly adopted throughout the world and are being constructed to ever-increasing heights. This Symposium has been published to provide a comprehensive and convenient reference work on rockfill dams. It is hoped that it will contribute toward improved, more economical, and higher rockfill dams of all types.

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[3] "The Design of Rockfill Dams", by J. D. Galloway, Transactions, ASCE, Vol. 104, 1939, pp. 1-92.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3063

ROCKFILL DAMS:
DIX RIVER DAM

By Lewis A. Schmidt, Jr.,[1] F. ASCE

With Discussion by Messrs. F. L. Lawton; and Lewis A. Schmidt, Jr.

SYNOPSIS

Constructed to a height of 275 feet in 1923-25 Dix Dam was then the highest rockfill dam built. Over a period of 33 years considerable data on leakage, maintenance, condition and settlement have been accumulated. This information is presented herein for applicable consideration in the design of similar future projects.

HISTORY

Dix Dam is a rock fill dam, which was built in 1923-1925 on the Dix River in Central Kentucky approximately 3 miles upstream from its confluence with the Kentucky River. It was built for the Kentucky Hydro-Electric Company, which later became the Kentucky Utilities Company, the present owner. The project was designed by Mr. L. F. Harza, the founder of the present Harza Engineering Company Inc. of Chicago. Consultants were Arthur P. Davis and M. M. O'Shaughnessy. A location map is shown in Figure 1.

Note.—Published, essentially as printed here, in June, 1958, in the Journal of the Power Division, as Proceedings Paper 1683. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Pres., Schmidt Eng. Co., Inc., Chattanooga, Tenn.

Design Features

The crest of the dam was built to elevation 780 with 5 feet of superelevation in the deepest section for settlement allowance. Normal river bed level was from elevation 505 to 515. The dam is 275 feet high in its highest portion, 1020 feet in length along the crest, with the upstream slope built on a vertical curve starting at 1 on 1.2 at the base and ending with 1 on 1 at the crest. The downstream slope is uniformly 1 on 1.4. The crest is 20 feet wide and follows a compound curve of 1700 and 10250 feet radii at the east and west sides respectively. The final volume of the rock fill was 1,756,000 yards of which 70,700 cubic yards were accounted for by the derrick laid rock section. A plan of the dam and spillway is shown in Figure 2 and details of the dam through the maximum section are shown in Figure 3.

A spillway channel, 250 feet wide, was designed to be cut into the west or left bank, flaring out to a crest length of 800 feet at the actual spillway lip. Rock to construct the dam was obtained from this spillway cut which is 2000 feet long.

A 24 foot diameter, concrete lined, horseshoe shaped tunnel, 875 feet long, shown in Figure 4, was driven in the east cliff for the initial purpose of diversion during construction and for the ultimate purpose of carrying the water from the reservoir to the powerhouse. At the upstream end of the tunnel a chimney type intake tower was constructed and 125 feet upstream from the downstream portal of the tunnel a concrete plug, 35 feet long, was built to contain the entrance to three 8 foot diameter penstocks which connect to a 30,000 horsepower powerhouse of three equally sized hydroelectric generating units. A derrick laid rock section, 14 feet thick at the base, and 4.5 feet thick at the crest, was constructed as the base for a reinforced concrete diaphragm of 18-inch thickness at the cutoff trench and 8-inch thickness at the crest. A cutoff trench 8 feet wide and of depth to good sound foundation rock was excavated in rock and concreted back. Up to elevation 670 a 3-inch T and G timber face was left in place outside of the concrete diaphragm.

The original spillway consisted of a modified concrete rollway at elevation 750. Soon after operations began flashboards were installed on this spillway. In 1930 it was decided to raise the maximum operating level of the reservoir from 750 to 760. At this time 10 gates, 10 feet high and 35 feet clear opening were installed over a distance of 394 feet of the spillway. These gates were operated from above by two gantry cranes. The balance of the spillway, or 277 feet was raised by proposed collapsible concrete sections, which were designed to go out only in case of an unusual flood condition. By 1949 it was felt that the collapsible section should be replaced and an Ogee section at elevation 758.5 feet with 1 1/2 foot of flashboards was installed at that time.

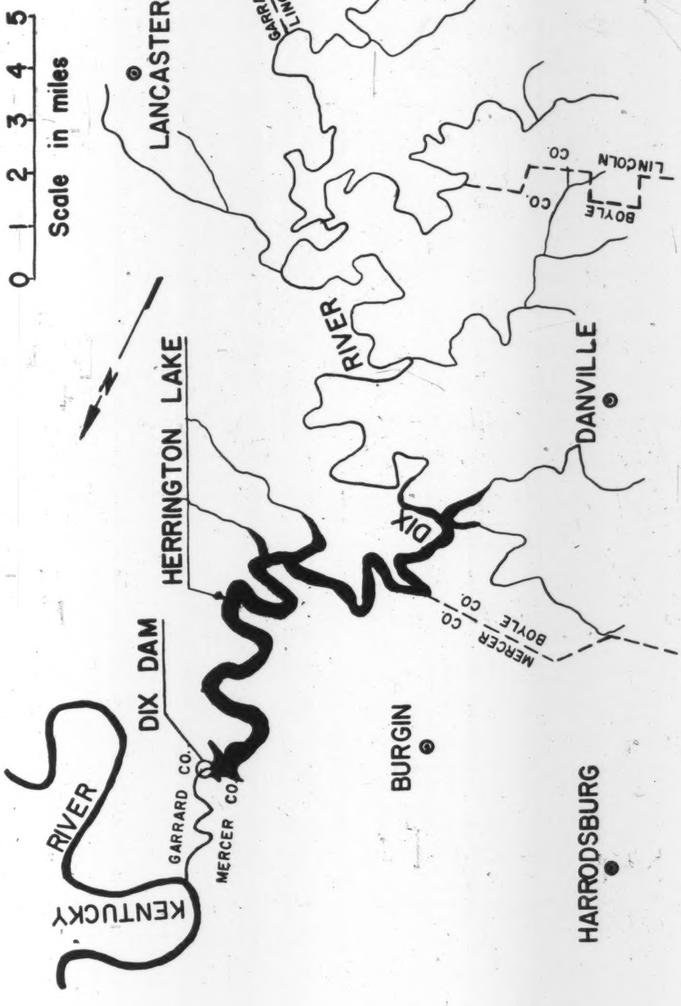

0
1
2
3
4
5
Scale in miles
N
LANCASTER
GARRARD CO.
LINCOLN CO.
HERRINGTON LAKE
RIVER
DIX
DANVILLE
LINCOLN CO.
BOYLE CO.
DIX DAM
KENTUCKY RIVER
GARRARD CO.
MERCER CO.
BURGIN
HARRODSBURG
MERCER CO.
BOYLE CO.

FIG. 1.—LOCATION PLAN

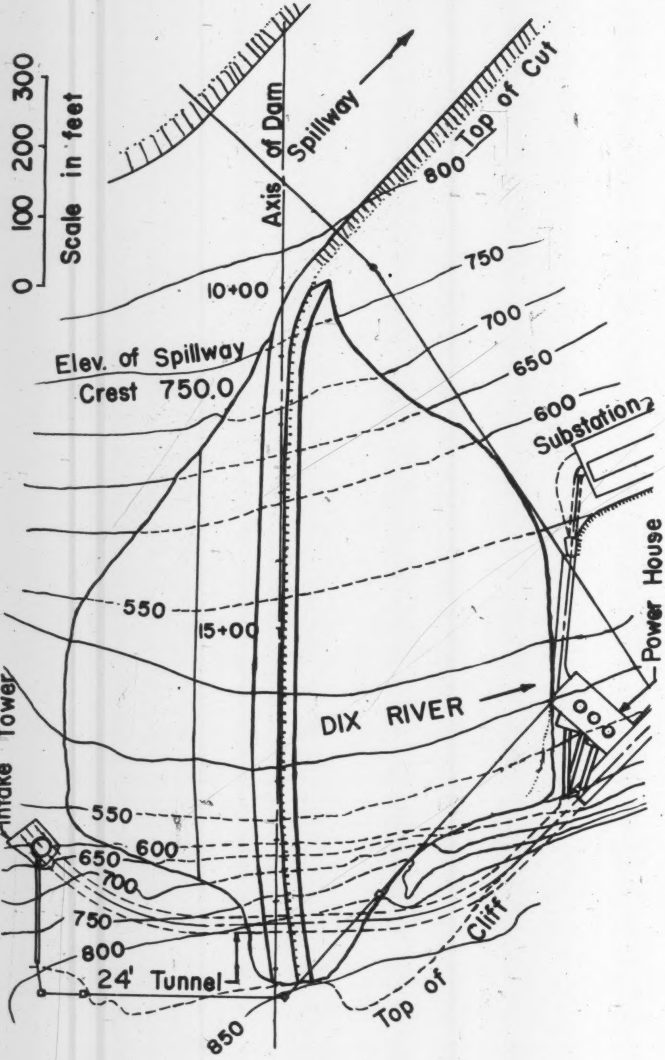

0
100
200
300
Scale in feet
Axis of Dam
Spillway
Top of Cut
800
750
700
650
600
10+00
Elev. of Spillway
Crest 750.0
Substation
Power House
550
15+00
DIX RIVER
Intake Tower
550
600
650
700
750
800
24' Tunnel
850
Top of Cliff

FIG. 2.—PLAN OF DIX DAM AND SPILLWAY

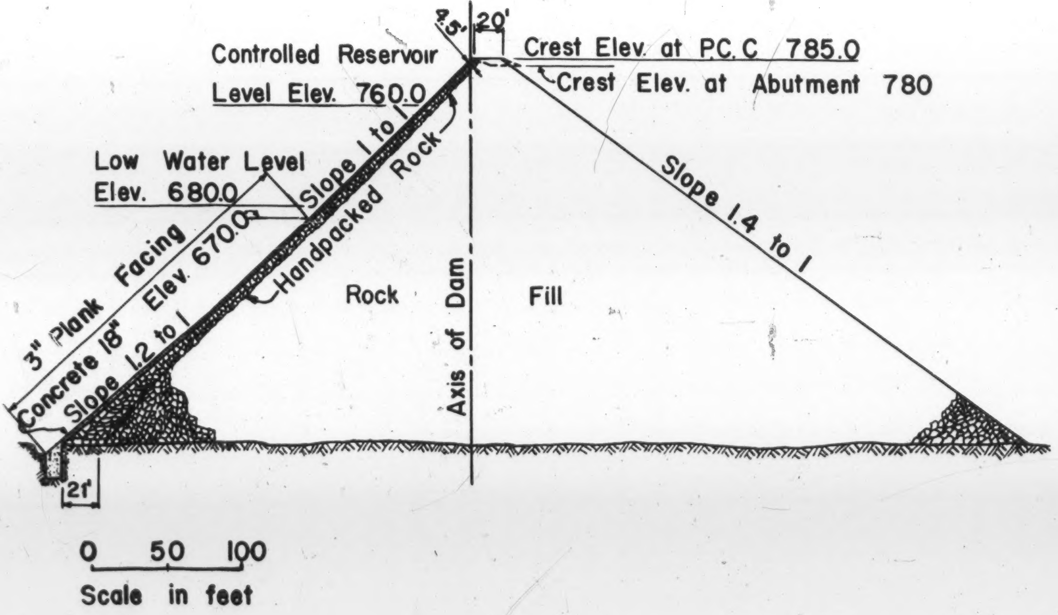
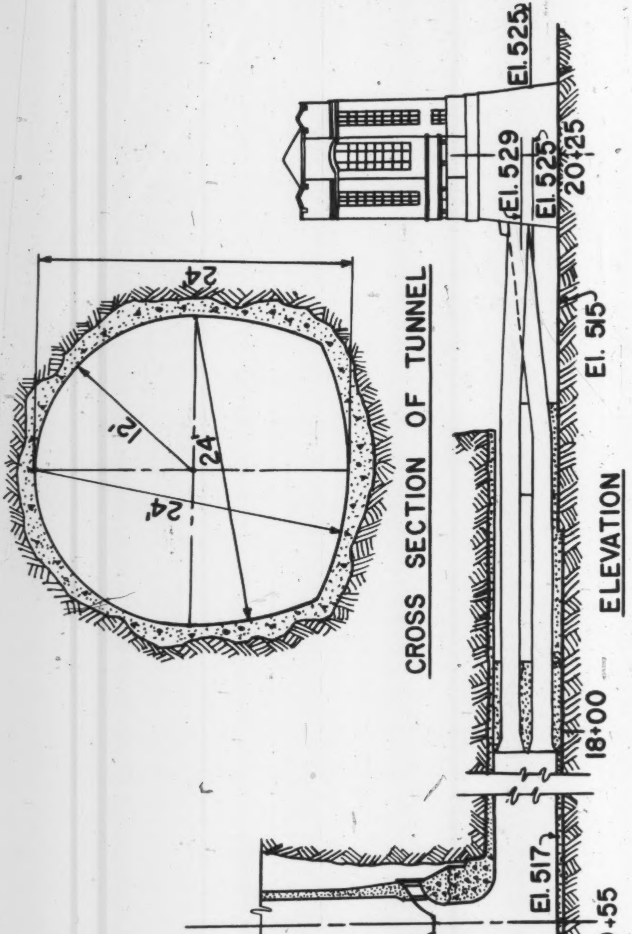

4.5'
20'
Controlled Reservoir
Level Elev. 760.0
Crest Elev. at P.C.C 785.0
Crest Elev. at Abutment 780
Low Water Level
Elev. 680.0
Slope 1 to 1
Handpacked Rock
3" Plank Facing
Concrete 18" Elev. 670.0
Slope 1.2 to 1
Rock
Axis of Dam
Fill
Slope 1.4 to 1
21'
0 50 100
Scale in feet

FIG. 3.—MAXIMUM SECTION


CROSS SECTION OF TUNNEL

ELEVATION

24'
12'
24'
24'
El. 525
El. 529
El. 525
20+25
El. 515
18+00
El. 517
10+55
El. 568.25

FIG. 4.—DETAILS OF TUNNEL

A section through this portion of the spillway is shown in Figure 5. Additional spillway capacity at the gates was provided by lowering the gate seat levels. The gates were made larger and reinforced to meet this additional head of water. Two gate seats were lowered 1 foot each, two were lowered 2 feet each and three were lowered 1 1/2 feet each with three remaining at the original level.

Construction Features

All talus material was stripped from the east bank and wasted. Similarly all the west side flood plain was stripped and the material wasted. This work was done with steam shovels and narrow gage railroad equipment. At the start of the job it also was planned to strip the entire spillway before loading out rock for the fill. In the interest of speeding up the work, portions of the stripping were omitted and this material was loaded out with the top cut of rock. Material in cars containing a majority of earth was wasted - other cars were dumped in the fill.

Rock excavation in the spillway channel was done by conventional drilling and blasting and rock loading was done with 3 1/2 cubic yard steam shovels of the railroad type which were converted to caterpillar mounting on outriggers at the front and a single rudder type, double caterpillar mounting at the rear.

Transportation was by 16 yard standard gage side dump cars, hauled by saddleback steam locomotives on a system of switchback railroad tracks over which trains traveled from the top of the west bank to the valley below.

Upon completion of overburden stripping from the base of the proposed dam, rock was side dumped from the west, or left bank switchback track at the 634 level, which was approximately 120 feet above the level of the river bottom. Figure 6 shows this operation. Side dumping and throwing of track was continued outward at this level until the fill reached almost across the canyon. At this time an 80,000 pound black powder coyote shot was pulled in the east cliff, just downstream from the downstream outline of the dam, which accounted for about 50,000 cubic yards of rock within the design limits of the dam. Later a well drill, dynamite shot of similar proportions was pulled in the east bank just upstream from the upstream outline of the dam and resulted in approximately the same volume of rock within the confines of the dam.

With this level at elevation 634 effected by side dumping, and the river diverted through the tunnel, a second lift of rock fill was then started. This lift was built by constructing a trestle above the 634 level to elevation 704 across the valley along the axis of the dam. From the level of this trestle rock cars were dumped for the full length of the trestle, the material being allowed to drop until the natural angle of repose had reached to the trestle level after which track was shifted both upstream and downstream until the fill met the required design outlines of the dam.

When the 704 level lift had been completed, a similar trestle was erected above this to elevation 774 and the rock fill was dumped from this level the same as from the lower level. The 774 level trestle is shown in Figure 7. The final crest elevations of 780 at the ends and 785 at the highest point were obtained by jacking track.

Throughout the filling operations, sluicing monitors were used to sluice

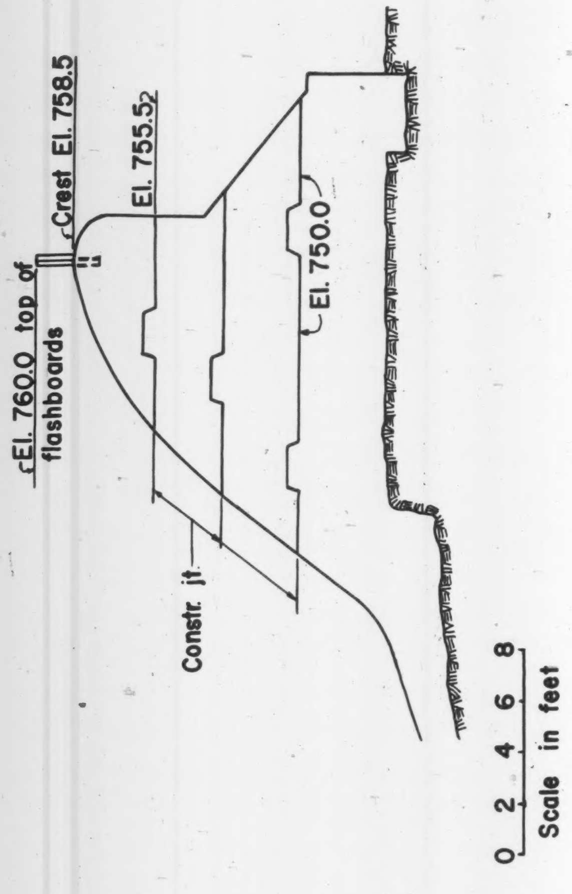

El. 760.0 top of flashboards
Crest El. 758.5
El. 755.5
El. 750.0
Constr. jt.
0
2
4
6
8
Scale in feet

FIG. 5.—DETAIL OF SPILLWAY

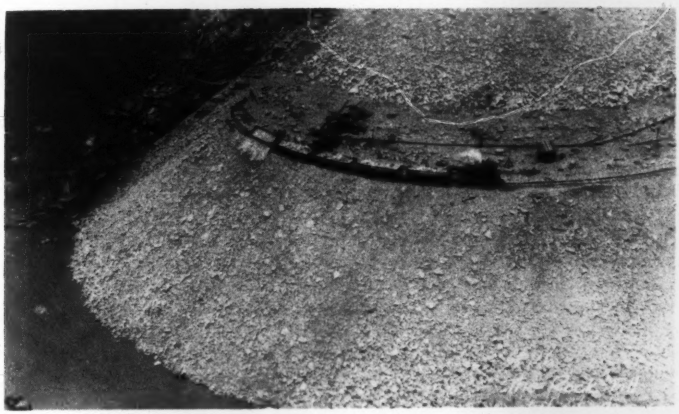


FIG. 6.—SIDE DUMPING FROM WEST SIDE

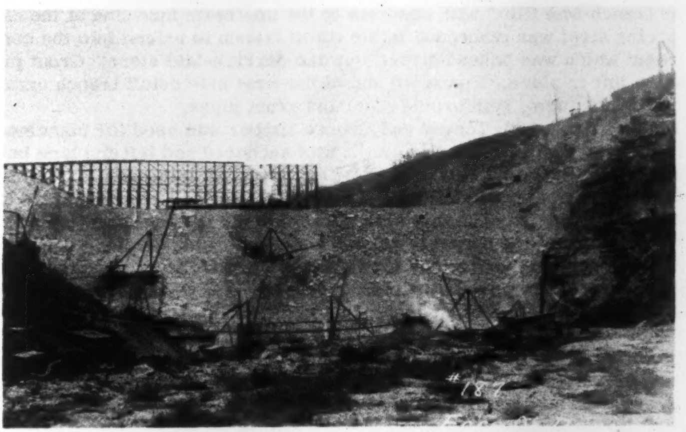


FIG. 7.—UPSTREAM VIEW SHOWING LEVEL COMPLETED AND START OF TRESTLE

down the fill. However candor must concede that this operation by comparison with the volume of fill placed was probably not as effective as it was intended to be.

All rock was hauled from the spillway area except that produced by the two east side shots which accounted for hardly more than 100,000 cubic yards and a small amount from an east side quarry which was soon abandoned as an uneconomical operation. Except to hold a 250 foot channel width and an 800 foot lip length, the spillway grades were varied to suit the amount of rock required to complete the fill and surpass the gradient required to pass floods.

Simultaneously with placement of the fill, the derrick-laid section of the dam was being constructed. Rocks weighing from 3 to 10 tons each were selected from the fill and hauled to the section by cables and chains from derricks set up along the face. These rocks were set to line and others of similar size were placed behind them, the entire section being chinked up with spalls and rock hammered into a tight mass. A section through this derrick-laid rock is shown in Figure 8. It was contemplated originally that a sufficient supply of large rock would be available from the spillway excavation to build the entire derrick-laid section of the dam. However a shortage of face rock of adequate size became apparent when about half the face had been constructed. Rock in roughly 3 to 5 foot cubes was imported from the Bedford, Indiana limestone quarries for facing purpose, same being culls from the building stones quarried there.

The cutoff trench was excavated in the limestone rock to a level of seemably hard and tight rock in accordance with the plans as depicted in Figure 9. The trench was filled with concrete to the upstream face line of the dam. Reinforcing steel was embedded in the cutoff trench to extend into the concrete diaphragm which was poured directly on the derrick-laid stone. Grout pipes were also left in place. Figure 10 shows the west side cutoff trench excavation, initial top form, reinforcing steel and grout pipes.

A top form of 3-inch Tongue and Groove timber was used for placement of this reinforced concrete diaphragm. It was anchored and left in place between the cutoff wall and elevation 670, the estimated minimum level of the drawdown. Above elevation 670 the timber was removed after concrete had cured. Reinforcing steel in this slab was 1/2 of 1% in each direction. Figure 11 shows details of the concrete diaphragm, reinforcing steel and timber sheeting. Concreting operations for this face slab are shown in Figure 12.

Vertical recesses were built into the derrick-laid rock, 36 inches wide by 18 inches deep, and 48 feet on centers as depicted in Figure 13. These were filled with concrete and include an 18 inch rib protruding upward the depth of the face slab. These ribs were heavily reinforced and equipped with expandable copper water stops extended into the slab with 1 inch cork board between the rib and the slab and with tarred felt over the bearing surface of the sleeper. Horizontal expansion joints were similarly built at elevations 550, 600, 650 and 700 as shown in Figure 14. These consisted of recesses in the rock 30 inches wide by 16 inches deep, filled with concrete, well reinforced and with a tarred felt over the sleeper over which the face slab rides. A copper seal connects the face slabs and 1 inch cork board separates the seal. Concrete slabs were therefore 48 feet by 71 feet of varying thicknesses and waterstop joints between the ribs and the slabs were of 18 gage copper sheets. Figure 15 shows the recesses for the vertical and horizontal joints in the derrick-laid masonry.

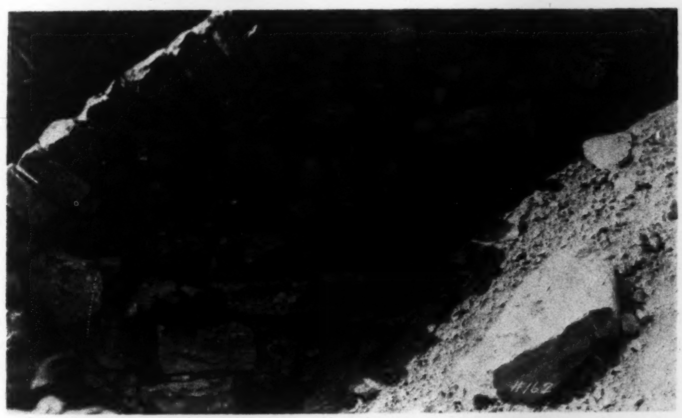


FIG. 8.—SECTION THROUGH DERRICK-LAID ROCK MASONRY

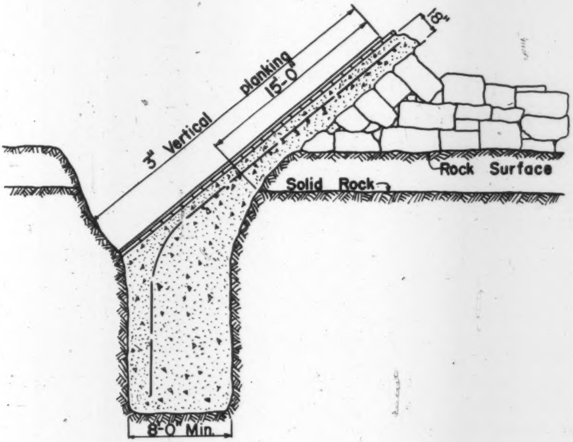

3" Vertical planking
15'-0"
18"
Rock Surface
Solid Rock
8'-0" Min.

FIG. 9.—DETAIL OF CUTOFF WALL

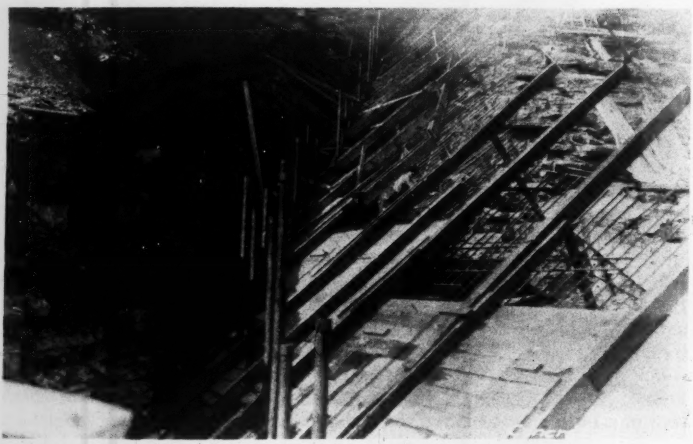


FIG. 10.—CUTOFF TRENCH CONCRETING OPERATIONS

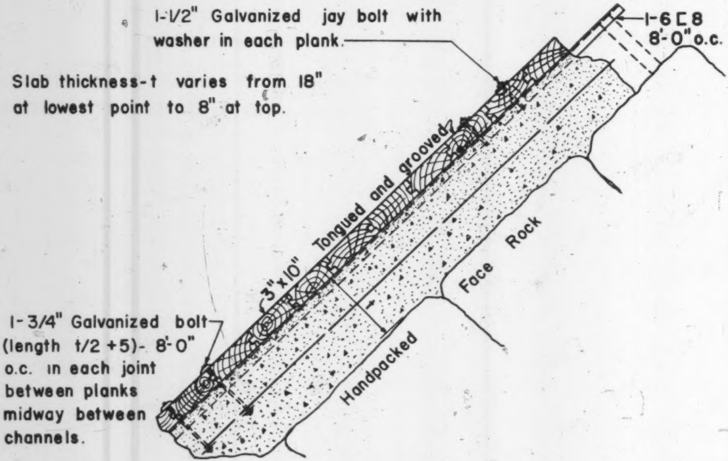

1-1/2" Galvanized jay bolt with washer in each plank.
1-6 [8 8'-0" o.c.
Slab thickness-t varies from 18" at lowest point to 8" at top.
Tongued and grooved
3"x10"
Face Rock
Handpacked
1-3/4" Galvanized bolt (length t/2 +5)- 8'-0" o.c. in each joint between planks midway between channels.

FIG. 11.—CONCRETE FACE SLAB

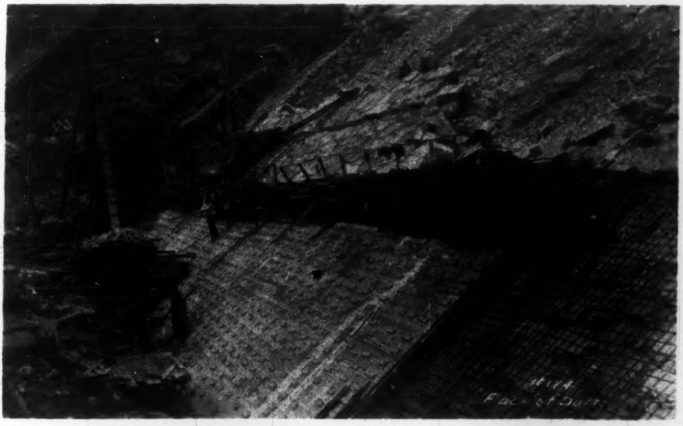

#174
Face of Dam

FIG. 12.—CONCRETING OPERATIONS ON FACE SLAB

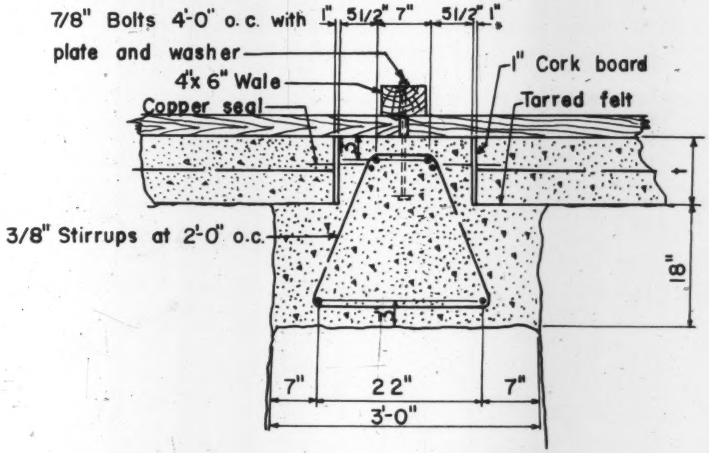
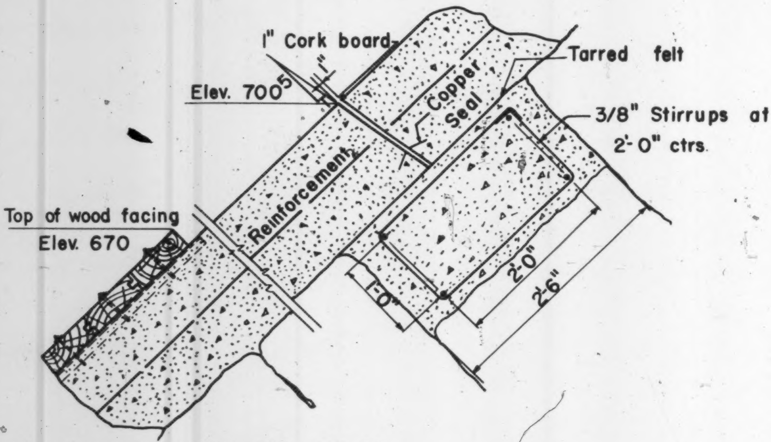

7/8" Bolts 4'-0" o.c. with plate and washer
4"x 6" Wale
Copper seal
1"
5 1/2"
7"
5 1/2"
1"
1" Cork board
Tarred felt
3/8" Stirrups at 2'-0" o.c.
1'
18"
7"
22"
7"
3'-0"

FIG. 13.—DETAIL OF VERTICAL EXPANSION JOINT


1" Cork board
1"
Elev. 700.5
Copper Seal
Tarred felt
3/8" Stirrups at 2'-0" ctrs.
Reinforcement
Top of wood facing
Elev. 670
1'-0"
2'-0"
2'-6"

Joint shown is at elev. 700. Planking is continuous over other joints.

FIG. 14.—DETAIL OF HORIZONTAL EXPANSION JOINT

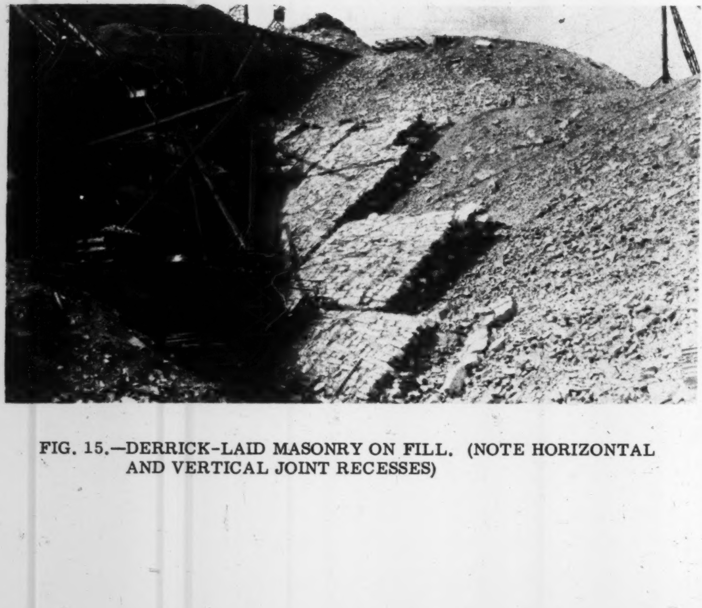


FIG. 15.—DERRICK-LAID MASONRY ON FILL. (NOTE HORIZONTAL AND VERTICAL JOINT RECESSES)

Foundation Treatment

The site is geologically situated in the Inner Blue Grass Region. The Dix and Kentucky Rivers have cut deep narrow gorges in lime-stone and shales of the Black River Ages. The oldest Black River beds are exposed in the bed of the river and for a distance of about 50 to 75 feet up the side of the gorge. The unit consists of pure, thin-bedded, dove-colored limestones with thin interlayers of calcareous shale and two thin layers of volcanic clay or bentonite. Younger Black River rocks are present in the higher parts of the abutments and overlying Trenton rocks are beneath the surrounding upland. This unit is composed of light to dark blue-gray limestone with thin interbeds of gray shale.

Structurally the rocks are generally flat lying but with a gentle southward regional dip and only local open folds break this general trend. Two sets of joints are prominent which tend to divide the bedrock into cube-like blocks. Both sets of joints have nearly vertical dips. The north-south trending set is the most conspicuous at the damsite since its trend is approximately parallel to the river and the individual joints are longer than the east-west trending set.

Weathering along such structural features as bedding planes and joints are the only defects in the bedrock at the site. The limestones at the site have been exposed to weathering for a long time and therefore a system of bedding plane seams and open vertical joints has been developed to great depths in the abutments and to moderate depths in the river bed. Most of these seams and channels are largely filled with a residual cave clay left after the solution of the impure shaly limestones and it is likely that the clay filling has washed out of many of these seams.

Because the foundation rock was limestone susceptible to solution channels and cavitation, a drilling and grouting program was pursued. Holes were 3 inches in diameter and 8868.4 feet were drilled. These were grouted using 4794 cubic feet of grout. By present day standards this program was probably insufficient but it certainly was a substantially improved operation at the time. Shot drills, which were slow and cumbersome, were used. Thirty-one (31) holes 40 feet on centers were drilled averaging 215.3 feet deep and grouted with air pressure type grout machines. Twelve (12) intermediate holes, 20 feet on centers with the primary holes, averaging 109.1 feet were drilled and grouted. Eleven (11) other intermediate holes bringing the spacing to 10 feet and averaging 71 feet deep and 3 five foot center holes averaging 34.4 feet deep were placed in areas of large grout takes in the primary and secondary holes in order to build up a grout curtain throughout. All holes were drilled under the cutoff trench and under the spillway structure. Certain horizontal seams were found and these largely determined the depths of the intermediate holes.

Because of the number of open crevices, grouts made up from ordinary concrete sand and cement were used in some cases and it is probable that these grouts often did not fill the openings completely as was intended. It is also likely that a certain amount of segregation was experienced. Grouting procedure was to run the first 50 bags of cement with mixes of 1/2 cubic foot of cement to 15 1/2 gallons of water. The next 50 bags of cement were run in mixes of one cubic foot of cement per 15 1/2 gallons of water. Thereafter sand was added in mixes 1 cubic foot of cement to 1/2 cubic foot of sand and finally 1:1 until the hole closed off. Holes were grouted from a single setting at the top of the hole. A diagram of the original grout holes is shown in Figure 16.

Leakage

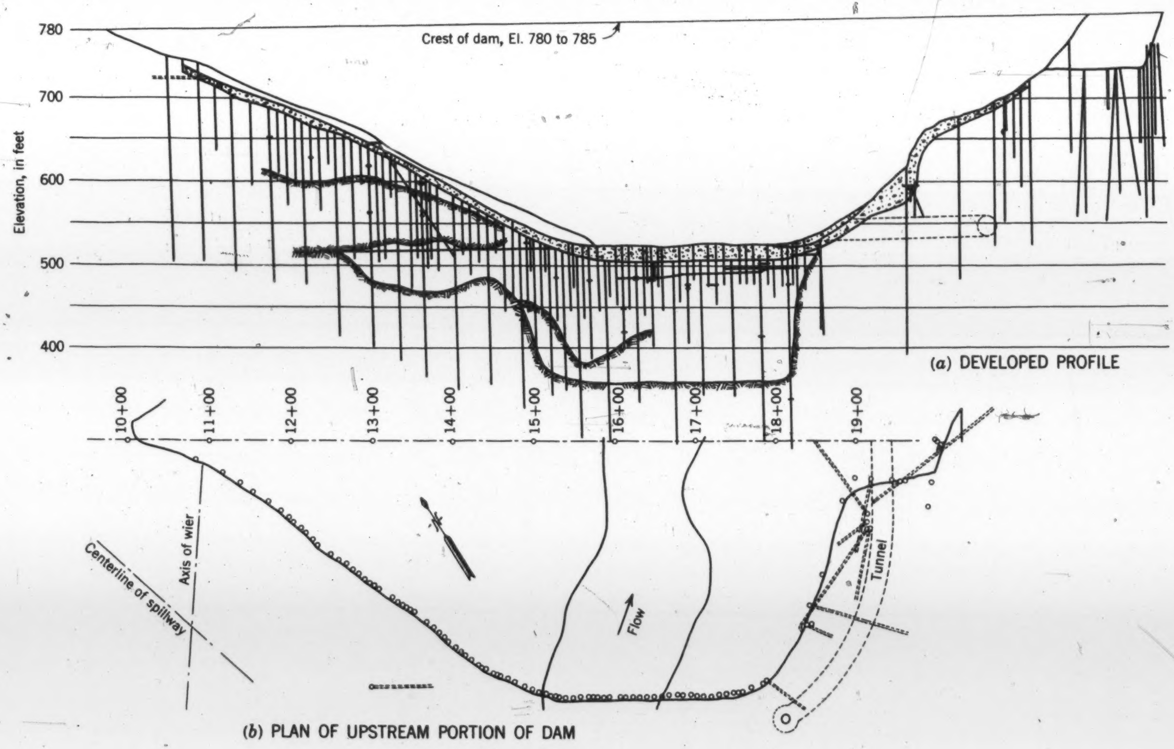
With the fill at elevation 774, derrick-laid rock at elevation 634 and above, and the concrete apron at elevation 634, the gate in the intake tower to the diversion tunnel, Figure 17, was closed and water from a general rain was stored for initial operations. Figure 18 shows the project just before closure. Shortly thereafter certain leakage was found to exist at the downstream toe of the dam near the river level. The water was generally clear and presented no apprehensions insofar as dam stability was concerned. This leak still persists even some 33 years later. As the water level in the reservoir was raised certain other leaks have also developed along the east and west banks and one at the downstream end of the tunnel.

At various times between 1925 and 1944 drilling and grouting operations were undertaken, using cement grouts, and some asphalt grout on a small scale. The sources of leakage were explored with audiophone equipment and dyes. One source of substantial leakage was indicated at a comparatively low level near the intake tower by the sonic device. Other sources were found along the face of the dam at the joints of the slabs and the ribs. Large quantities of cinders were dumped on the face and in the corner near the intake in 1936 and in a measure showed a marked temporary improvement in the leakage,

However by 1944 the main leak, measured by a weir adjacent to the powerhouse, showed leakage to be 82 cfs under 235 feet of head. Other leaks under 212 foot head were measured coming from the tunnel and east and west banks. These were 29 cfs, 8.5 cfs and approximately 15 cfs respectively. Sources of this leakage had been traced to some areas along the east cliff near the intake tower and, although not firmly established, there was evidence that some leakage originated from the west bank and through the apron of the dam.

At the same time (1944) it was decided to attack the foundation leakage problem in orderly fashion and several patterns of holes were laid out in the spillway area for the purpose of establishing possible leakage paths through the west abutment. These holes were drilled to levels below the riverbed and dye tested for leakage. Some of them showed leakage connections, the dye showing along the west bank of the river downstream from the powerhouse and at the weir. These holes were grouted with cement and asphalt as applicable.

Another pattern of holes was laid out on the east bank, drilled, tested and similarly grouted. A few of these holes showed leakage to the weir and to points along the east cliff at low levels. Measurements were made at the weir, at intermediate places between the weir and a riffle approximately 1200 feet downstream, at this riffle, at intermediate places between the riffle and a cableway measuring station approximately 1800 feet downstream from the riffle and at the cableway station. Records of the leakage measurements by dates and existing heads were made and the percentages of decreased leakage at all of these stations for different increments of head were computed. For heads of from 220-230 feet, the average decrease was 20% as a result of work done during the period between 1944 and 1956. For heads between 210-220 feet, the decrease was 29%. For heads 200-210 feet, leakage was reduced by 18% and for heads between 184-200 feet the decrease was 38%. In practically


Crest of dam, El. 780 to 785
780
700
600
500
400
Elevation, in feet
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00
18+00
19+00
Axis of wier
Centerline of spillway
Flow
Tunnel

(a) DEVELOPED PROFILE

(b) PLAN OF UPSTREAM PORTION OF DAM

FIG. 16.—ORIGINAL DRILLING AND GROUTING

DETROIT PUBLIC LIBRARY

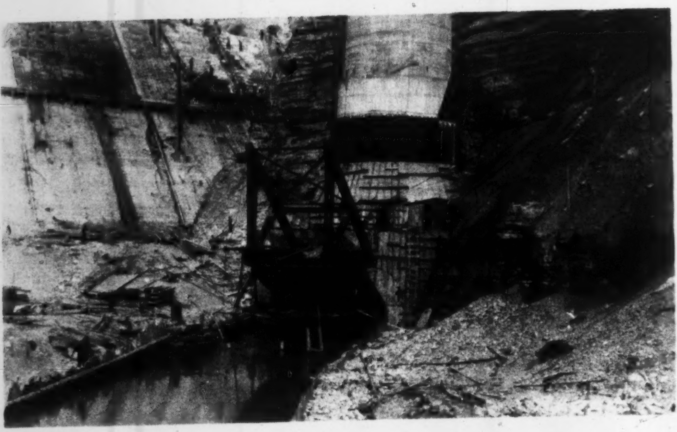


FIG. 17.—INTAKE TOWER CLOSURE GATE. (NOTE EAST SIDE CUTOFF WALL)

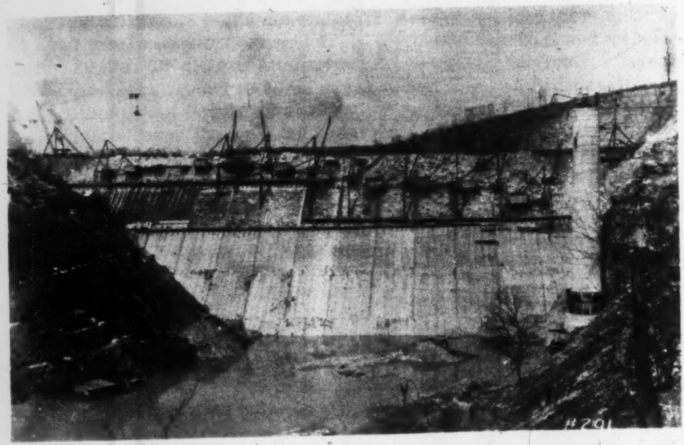


FIG. 18.—UPSTREAM VIEW OF DAM JUST BEFORE CLOSURE

each case the extremes were not too far removed from the norms. Table 1 is self explanatory.

TABLE 1.—PERCENTAGE REDUCTION IN LEAKAGE, 1944-1957

Head Differential, in Feet	Measured at Weir	Accounted for at Riffle	Measured at Riffle	Accounted for at Cableway	Measured at Cableway	Average
220-230	20	25	10	31	15	20
210-220	29	30	-	32	35	29
200-210	15	21	18	25	12	18
184-200	40	40	33	39	40	38

The only continuous leakage measurement over a range of heads is available at the weir which is also the predominant leak. A graph showing the effect of the drilling and grouting work is shown in Figure 19.

As this work progressed it became obvious that some type of maintenance program should be established to arrest this leakage. Even if the leakage could not be completely eliminated at once a continuing grouting program should be successful in the ultimate correction of this leakage and in the meantime its effectiveness in reducing the leakage or at least keeping it from increasing was recognized. This was done and the work continues. The results are at times very positive - at other times they indicate only that a holding action has been established. In any case, the effort and the expenditures seem amply justified because an examination of the records and results indicates that without this work the leakage was prone to have increased by greater proportions than the reductions actually affected. The effectiveness can therefore not be measured accurately in dollars and cents but the need for the work is an established fact.

In 1944 certain leakage was coming from the tunnel. This increased year by year until the leakage at the downstream tunnel portal had reached such proportions in 1949 that it was decided to undertake some remedial work at this end of the tunnel, working downstream from the penstock plug. The work had hardly begun when the water under pressure broke out a portion of the rock below the invert and to the east of the concrete tunnel lining just upstream from the concrete plug that holds the penstocks. It was necessary to close the intake tower gate and to discontinue operations while this major repair was being made. However leakage in this location had no bearing on the other leakage correction work except that the leak developed in a bentonite seam near the invert on the east side of the tunnel just upstream from the penstock plug.

This repair consisted of clearing out all debris, rocks and other accumulations, replacing the lining and filling the cavity with concrete. Furthermore the plug was lengthened 29 feet upstream with formed concrete penstocks in this new section connecting to the existing steel penstocks. The entire plug section, now 64 feet long was then drilled radially with holes 30 feet into rock, 16-18 holes being spaced approximately equally around the periphery of the tunnel and 7-10 feet on centers longitudinally. These holes were drilled from inside the penstocks and grouted to refusal with neat cement grout under 60#/sq. in. pressure immediately after each was drilled. A total of 119 holes were drilled to saturate a circle 84 feet in diameter from the centerline of the tunnel, and 2360 cubic feet of grout were used for this operation.

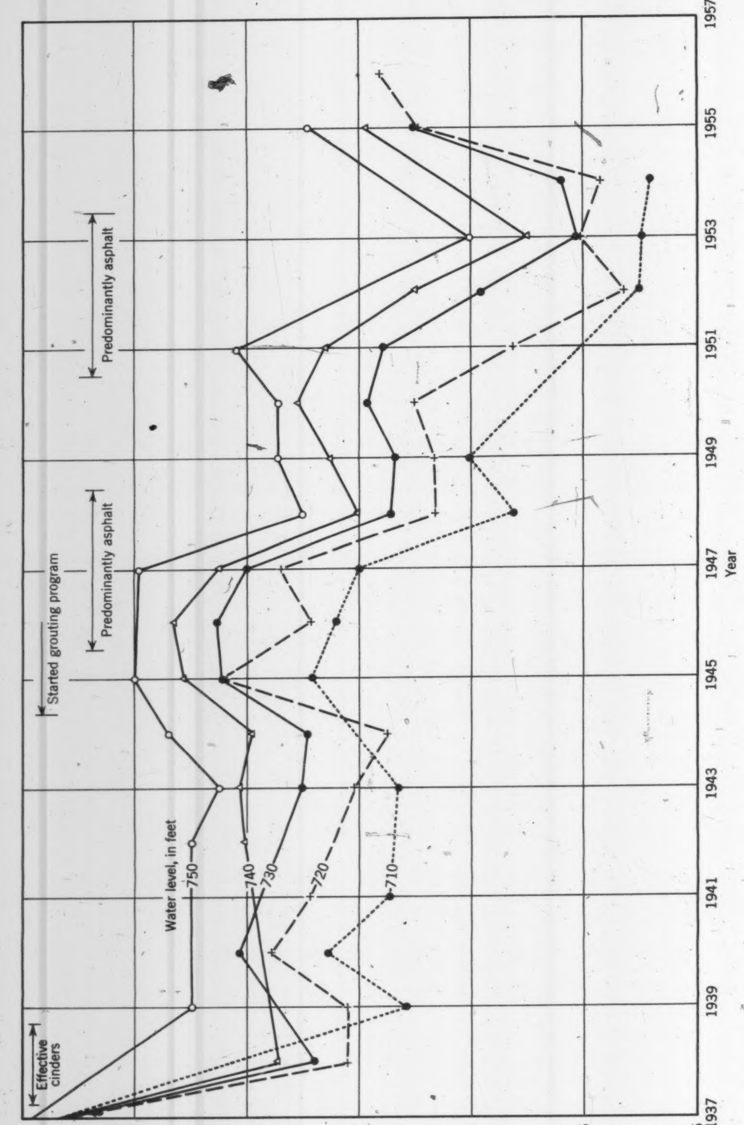


FIG. 19.—DIX WEIR LEAKAGE

The repair has been highly satisfactory and since it was completed there has been no leakage at the downstream tunnel portal.

Settlement

Figure 20 shows the derrick-laid section and the concrete apron nearing completion to the crest of the dam. At the time of construction a series of copper pins was established in the face concrete at the crest and at elevation 725 to record settlement of the dam. From time to time readings of the elevations and horizontal positions of these pins were taken. A limited number of readings could be obtained at elevation 725 due to the water level in the reservoir.

The significant point about settlement and lateral movement of the fill downstream is that it is still taking place after 32 years at both the crest and at elevation 725. Although there exist a few inconsistencies in the settlement readings, the trend is uniformly along parabolic curves. The inconsistencies are of such a minor nature as to be readily accounted for in spot readings of the field measurements.

An upstream elevation of the dam and actual elevations at the crest are shown for various years in Figure 21. This figure also shows the relative vertical settlement at four typical stations for readings taken both at the crest and at Elevation 725. Differential settlement and horizontal displacement at the crest are shown in Figure 22. The readings that could be obtained to show vertical settlement and horizontal movement at Elevation 725 are shown in Figure 23 which also shows the relative horizontal displacement of the dam at two typical sections for readings taken at the crest and at Elevation 725.

Some general deductions may be made from the settlement records.

1. As expected, the greatest settlement (and downstream movement) has taken place in the deepest sections. At the crest measuring stations, a maximum vertical settlement was 4.22 feet from 1925 to 1957 or 1.57%. The maximum downstream displacement at this level is 3.19 feet during the same period, or 75.5% of the vertical settlement.
2. Judging from the shape of the settlement curves, additional settlement in reduced increments will continue over the next 10 to 20 years. Having superelevated the dam by 5 feet in the deepest section when it was constructed was therefore a very good estimate of the settlement expectancy.
3. Settlement and movement downstream at the 725 level is less than at the crest (785-780) as was to be expected. Also incremental settlement and downstream displacement are less than at the crest after the 32 years, another anticipated feature. Maximum vertical settlement at this level during this period of time is 3.60 feet or 1.71% and corresponding maximum horizontal displacement is 2.60 feet, or 72% of the vertical settlement.

In general it can be said that the settlement and horizontal displacement behaviors are rational although extending over a much longer period of time than what perhaps was originally envisioned. However since the original superelevation was 1.85% of the height of the dam, it will still be some time before the settlement catches up with the depth of settlement originally provided for it.

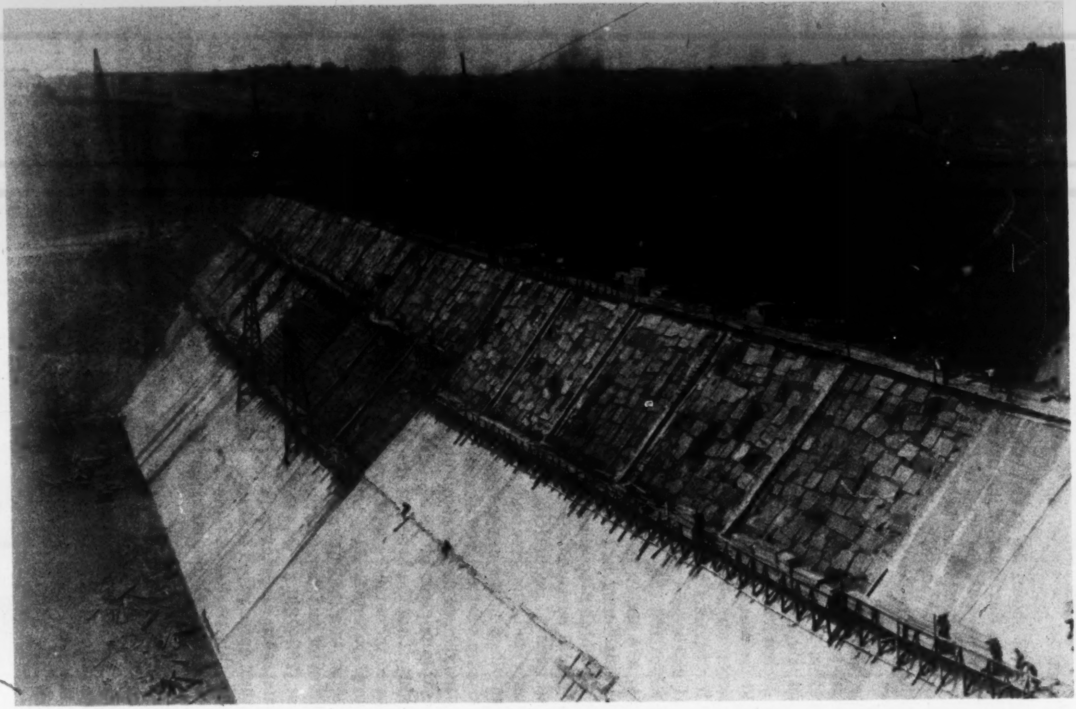


FIG. 20.—PROJECT NEARING COMPLETION

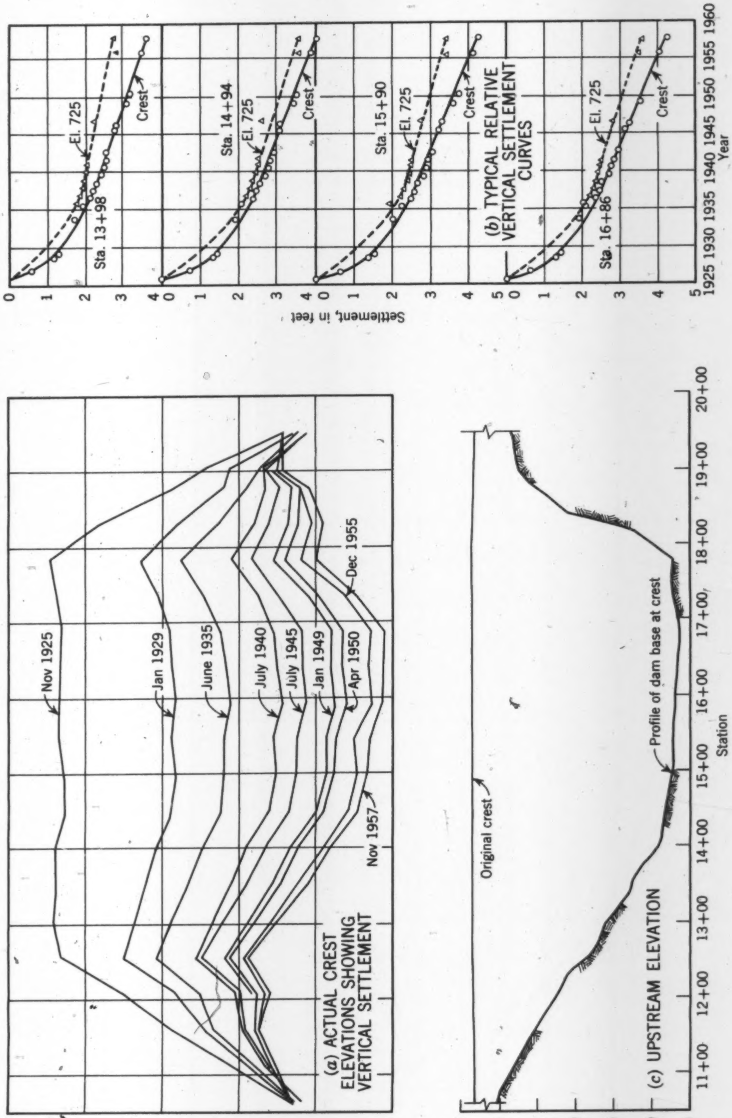


FIG. 21.—SETTLEMENT RECORDS

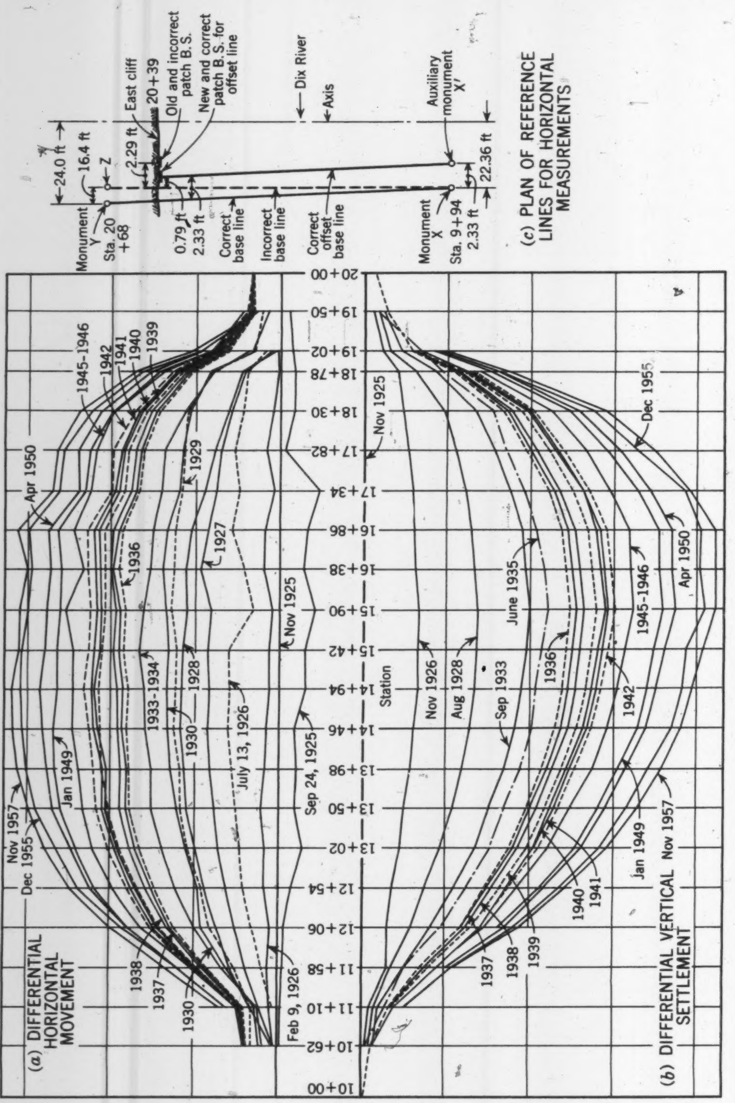


FIG. 22.—READINGS AT CREST

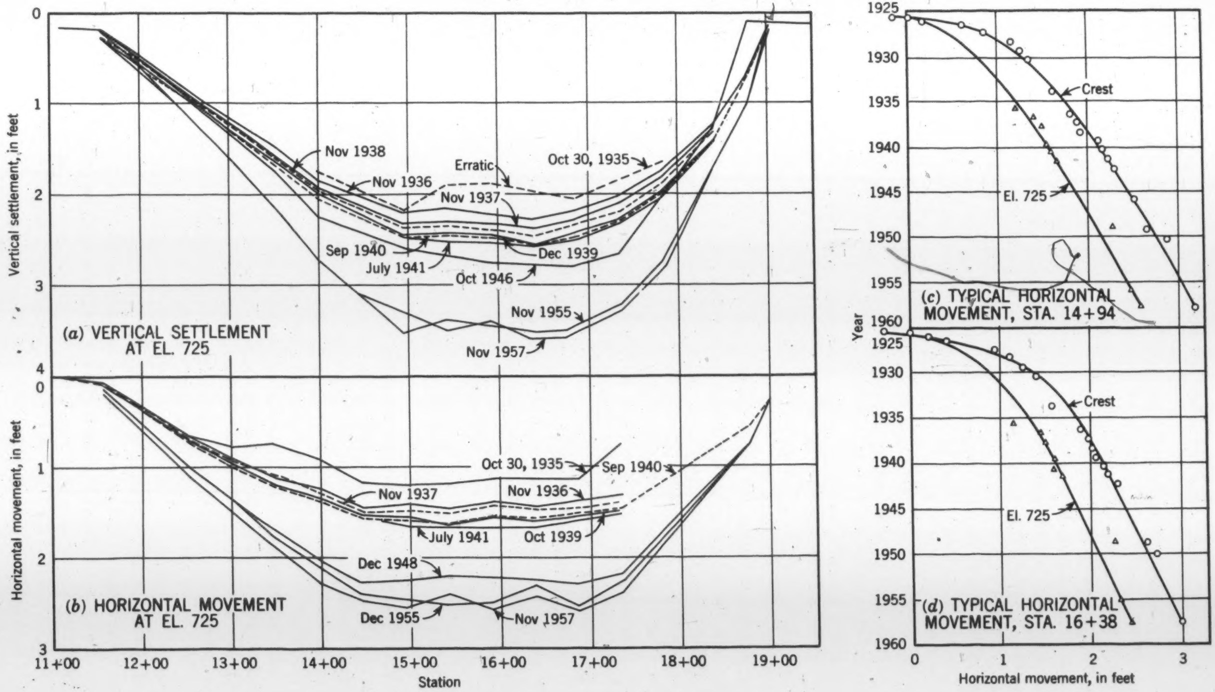


FIG. 23.—READINGS AT ELEVATION 725

Face Slab and Derrick-Laid Masonry

There is no way to ascertain the condition of the concrete face slab below low water level, elevation 670, without draining the reservoir. Similarly no inspection of the condition of the derrick-laid masonry section is possible because it is covered by the face slab.

In recent years the lowest water level attained has been elevation 696 in the fall of 1946 and between September and December of that year, detailed inspections of the face slab and particularly the vertical and horizontal joints were made between elevations 725 to 696 as the water level dropped.

The results of this inspection indicated numerous breaks in the vertical and 700 horizontal joints. The copper seals were pulled loose in a number of places. The concrete ribs forming the vertical joints were broken in a number of cases at the lower levels and there was audible leakage below elevation 696 in a substantial number of joints. Also there were quite a few cracks in the face slab varying from hairline cracks to open cracks, the latter occurring along the east cliff where vertical movement is augmented by horizontal movement both downstream and along the axis due to the steepness of the east cliff.

The greatest significance of this investigation was that the face condition was more seriously impaired progressively at the lower levels where water pressures and weight of the structure are greater. Conversely the upper portions of the face are exposed to greater temperature changes when exposed to the sun and the weather than those areas below water level. However the vertical and horizontal movements of the dam cause the slab to pivot around horizontal joints and the joint between the face slab and the cutoff wall so that some opening of these joints to potential leakage is not only possible but also highly probable.

While not of any immediate hazard nor an economic liability, it is recognized that some time in the future, the plant will have to be shut down and the face completely repaired. At such time a more complete and up-to-date grouting program under the cutoff trench should also be undertaken.

Reservoir Silting

Only one reservoir silting survey has been made in the Dix Reservoir. It was made in 1941 by the Conservation Service of the U. S. Department of Agriculture. Over the 16 years of reservoir life in 1941, the reservoir volume had been decreased by 1.25% or an average of 0.08 percent per year. Because only one measurement has been made, it is by no means certain that this rate will be maintained. Other measurements would be helpful in establishing the silting rate for this reservoir.

CONCLUSIONS

Built at a cost of approximately $7,000,000, the project has been an economic success, averaging an output of approximately 49 million kwh per year. Originally it was used as a base load plant but in recent years as larger steam generating stations began replacing hydro stations as base load plants, the value of this project has become even greater as a peaking station which is its present function in the system operations.

Dix Dam causes the impounding of water for hydroelectric energy purposes. Furthermore the City of Danville, Kentucky, a State Hospital and the E. W. Brown steam generating station are dependent on the lake for their water supplies and numerous fishing and recreational camps have been built on the shores of the lake. For these reasons it is not readily feasible to drain the reservoir, inspect and repair the face and do remedial foundation treatment under the cutoff trench. To that end the choice of a faced rock fill dam in a limestone region for power and other multi-purposes should probably be discouraged although for other purposes, where the reservoir can be drained periodically, or at locations where there is no danger of foundation leakage, a rock fill dam should function satisfactorily and in all likelihood can be built economically.

ACKNOWLEDGMENTS

The writer is indebted to the Kentucky Utilities Co. for permission to use the data published herein. It has been assembled by the writer over a period of years with the cooperation of Messrs. R. M. Watt, Chairman of the Board of Directors, E. W. Brown, Vice President until his death in 1957, W. A. Duncan, Vice President since 1957, Henry L. Johnson, General Superintendent of Production and Mr. G. D. Glass, Superintendent of the E. W. Brown Steam and Dix Dam Hydro Generating Stations, all of the Kentucky Utilities Co.

The writer also wishes to acknowledge the cooperation of the Harza Engineering Co., Inc. for making available prints of the original designs of the project.

DISCUSSION

F. L. LAWTON,[2] F. ASCE.—Reference is made in the paper to the two shots in the east cliff which each resulted in about 50,000 cubic yards of rock within the confines of the dam. Would the author care to indicate whether these mass movements of rock, although only a small part of the total volume of the dam, have contributed to leakage at the surfaces on which the movements were deposited?

The author's deductions from the settlement records are extremely significant, especially that indicating settlement and downstream movement may well continue over a period of some 50 years or more. This demonstrates the merit of constructing a high rockfill dam with a moderate convexity upstream.

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[2] Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

This continuing settlement and downstream movement can only occur through the crushing of point contacts in the coarser rockfill and a slow consolidation. It would be of interest to know the physical properties of the rock and if there has been any deterioration since the dam was built.

The experience at Dix dam demonstrates clearly the difficulty of securing a trouble-free concrete facing on a high rockfill dam.

It would add materially to the value of the paper if the author could indicate the total maintenance costs for the Dix rockfill dam since it was built.

The author is to be congratulated on a noteworthy contribution to engineering knowledge on rockfill dams, more particularly since the paper covers some 33 years of observation following construction.

L. A. SCHMIDT, Jr.,[3] **F. ASCE.**—Mr. Lawton in his discussion requests comment on whether the two large east cliff shots contributed to leakage at the surfaces on which the rock resulting from these movements was deposited. It is not likely that this is the case because practically all of the overburden in the deposited areas had been removed and the rock from the shots above fell or moved down with substantial impact that must have had some settling influence. It can be stated, however, that the sluicing operations were not as effective as they should have been for smaller lifts and therefore were much less effective for the greater volumes deposited momentarily.

Parenthetically, the large shots are believed to have had a much greater detrimental effect by virtue of the concussions created, which loosened blocky rock formations and opened potential paths for sidehill leakage. This effect was noticeable particularly in the tunnel after the blasts, where considerably more scaling was required, particularly in the crown of the arch than would have been necessary had the blasts not been pulled. This is known because the tunnel had been dug when the first blast was made but had not yet been concrete lined.

The physical properties of the limestone rock at Dix are as follows: It is a dense, compact finely crystaline, light gray rock. Its unconfined crushing strength would be in the range of 10,000 to 20,000 pounds per square inch. The small portion of silt and clay impurities forms a coating on the rock after long exposure giving it a characteristic drab gray color. There has been no apparent deteriation of the rock in the visible portions of the fill, nor in the abutting cliffs.

Unfortunately actual maintenance costs for this structure are not available. Except for the drilling and grouting and some minor patching of the concrete face, maintenance as such has required only minimum expenditures. However the tunnel repair occasioned a substantial expenditure all at one time when failure occurred. It is possible that this work could have been spread out over several years of maintenance and the obvious ultimate failure averted. However the need for electric energy during and immediately following the war years practically precluded such maintenance work because it would have meant shutting the plant down while the work was being done and every possible kilowatt of energy was needed for production until other planned generating could be constructed and placed in operation.

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[3] Pres., Schmidt Eng. Co., Inc., Chattanooga, Tenn.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3064

ROCKFILL DAMS:
COGSWELL AND SAN GABRIEL DAMS

By Paul Baumann,[1] F. ASCE

With Discussion by Messrs. W. A. Brown; John V. Spielman;
F. L. Lawton; Claude A. Fetzer and Elton G. Knight;
Harris Porter; and Paul Baumann

SYNOPSIS

Features of special interest attending the design, construction, and performance of two rockfill dams 280 and 380 feet in height, more or less, are treated in this paper. Particular attention is paid to such novel features in the design and construction as have proved to be advantageous and successful as well as those which, ingeniously, though they may have been conceived by the designer, were found to be undesirable, if not detrimental. The principal reason for the latter was the fact that conditions in the field failed to fully conform to those anticipated. Through frank discussion thereof it is hoped to avoid repetition and to thereby render a service to the profession.

INTRODUCTION

The San Gabriel and Cogswell Rockfill Dams of the Los Angeles County Flood Control District, formerly and perhaps better known as San Gabriel Dams No. 1 and No. 2, serve the dual purpose of flood control and water conservation. As shown in Fig. 1 they are situated on the main trunk and the West Fork of the San Gabriel River in Los Angeles County, California, which carries runoff from the San Gabriel Mountains to the sea. Their respective drainage areas comprise 163.5 and 39.2 square miles. They have the distinction of differing by 100 feet in height and 1000 feet in elevation above sea level, in round figures. Design and construction of Cogswell Dam preceded those of San Gabriel Dam. Hence, Cogswell Dam will be treated first. Furthermore, the performance of Cogswell Dam has so far only been treated

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Note.—Published, essentially as printed here, in June, 1958, in the Journal of the Power Division, as Proceedings Paper 1687. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Asst. Chf. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

in fragmentary discussions of a paper(2) whereas the performance of San Gabriel Dam was quite completely treated by the writer.(3)

To avoid confusion the original names of these dams, namely San Gabriel Dams No. 1 and No. 2, were adhered to in preparing the exhibits accompanying the text, in that the majority thereof were taken from widely distributed specification drawings of these dams.

Performance of Cogswell Dam

Exploration

Exploration of the foundation of the proposed dam was conducted by the Flood Control District through 7 diamond drill holes, 14 tunnels, 2 shafts, and 4 trenches. Respective information was made available to bidders. Through this exploratory work the general foundation characteristics were reasonably well established. Isolated variations therefrom which had to be anticipated under existing conditions at this dam site necessarily remained to be discovered and dealt with in the course of construction.

Design and Specifications

The specifications provided the material for the rockfill to consist of three classes A, B, and C of large rock with maximum size on the downstream face and toe, and with derrick placed rock, commonly known as packed rock, immediately below the facing. The character of all rock was to be sound, hard, durable, angular quarried rock, weighing not less than 160 pounds per cubic foot (p.c.f.); to be unaffected by air and moisture and of such toughness as to withstand dumping without undue shattering or breakdown; and to have a minimum compressive strength of 5000 pounds per square inch (p.s.i.).

Class A rock, for general use throughout the main fill of the dam, was to be a well graded mixture, 40 per cent of which to vary in weight from quarry chips to 1000 pounds, 30 per cent from 1000 to 3000 pounds, and the remaining 30 per cent from 3000 to 14,000 pounds. In addition, this mixture was not to contain more than 3 per cent of its total weight in quarry dust and the maximum dimension of any piece was not to be more than three times its minimum dimension.

Class B rock was to be selected extra large rock one-half of which to weigh not less than 14,000 pounds and the other half not less than 6000 pounds each. The greatest dimension of each piece was to be not more than four times its

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2. "The Design of Rock-fill Dams," by J. D. Galloway, M. ASCE, Transactions, ASCE, Vol. 104, 1939. Discussions by Cecil E. Pierce and Paul Baumann.
3. "Design and Construction of San Gabriel Dam No. 1," by Paul Baumann, M. ASCE, Transactions, ASCE, Vol. 107, 1942.

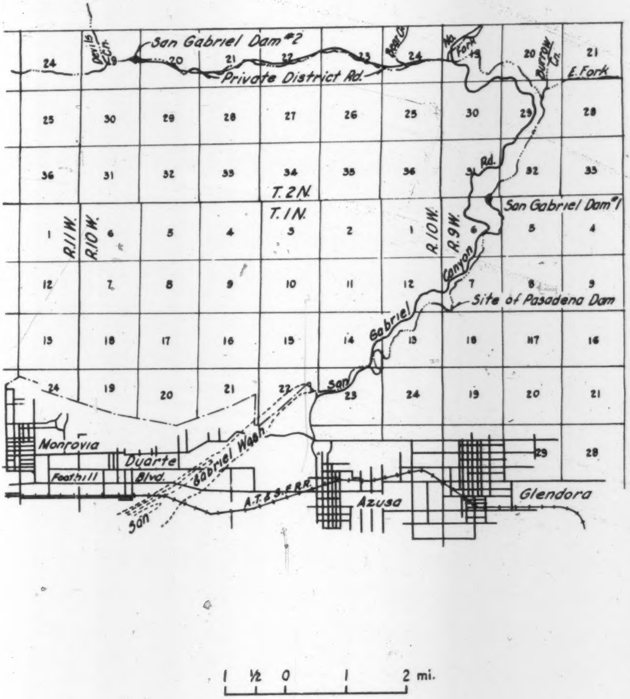
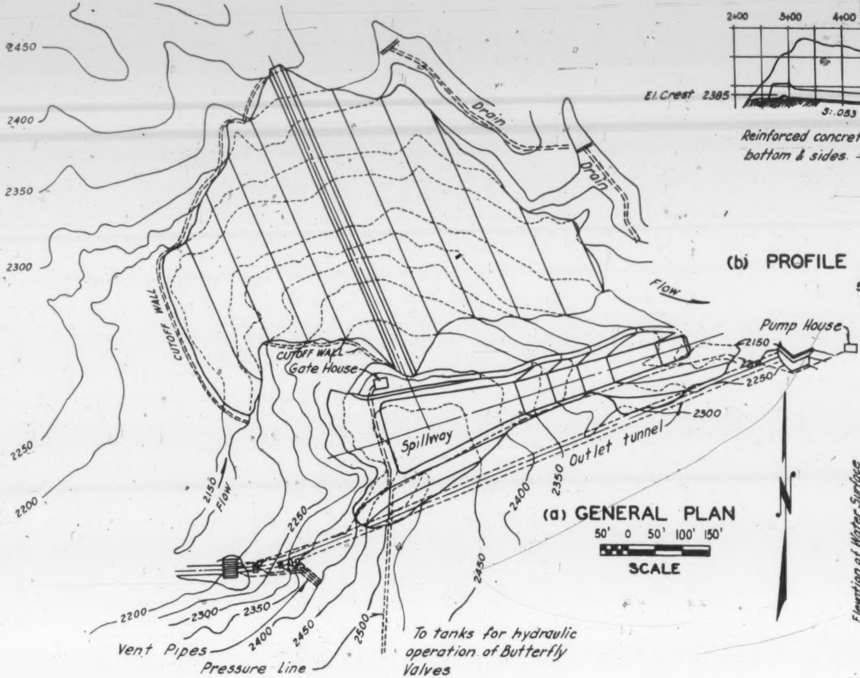
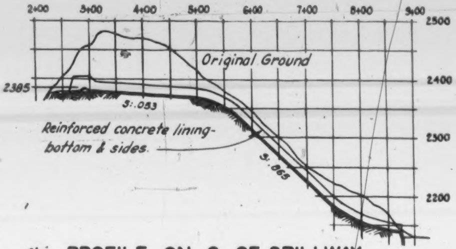

San Gabriel Dam #2
Devils Cr.
Private District Rd.
Bear Cr.
No. Fork
Burrow Cr.
E. Fork
Rd.
San Gabriel Dam #1
T. 2 N.
T. 1 N.
R. 11 W.
R. 10 W.
R. 10 W.
R. 9 W.
Canyon
Gabriel
San
Site of Pasadena Dam
San Gabriel Wash
Monrovia
Duarte
Foothill Blvd.
A.T. & S.F. R.R.
Azusa
Glendora
San
1 ½ 0 1 2 mi.

FIG. 1.—LOCATION MAP

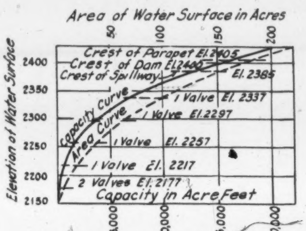

2450
2400
2350
2300
2250
2200
Drain
Drain
Flow
CUTOFF WALL
CUTOFF WALL
Gate House
Pump House
2150
2250
2300
Spillway
Outlet tunnel
2350
2400
2450
2500
2150
Flow
2250
2200
2300
2350
2400
2450
Vent Pipes
Pressure Line
To tanks for hydraulic operation of Butterfly Valves
50' 0 50' 100' 150'
SCALE

(a) GENERAL PLAN


2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
2500
2400
2300
2200
Original Ground
El. Crest 2385
S:.053
Reinforced concrete lining-bottom & sides.
S:.865

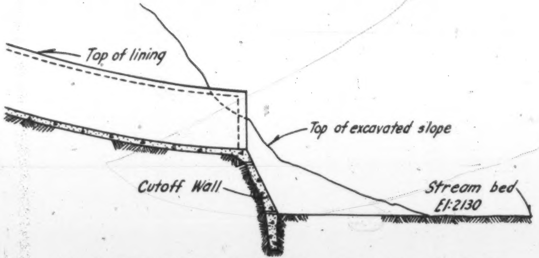
(b) PROFILE ON ℄ OF SPILLWAY

50' 0 50' 100' 150'
SCALE



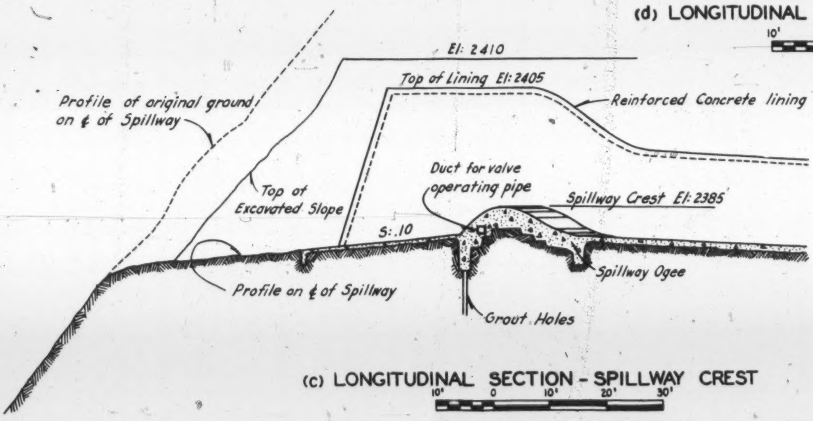
All Valves are 84" Dia. Butterfly Valves. El. given on ℄ of Valve

(e) CAPACITY - AREA CURVE


Top of lining
Top of excavated slope
Cutoff Wall
Stream bed
El: 2130

(d) LONGITUDINAL SECTION - SPILLWAY OUTLET

10'
0
10'
20'
30'


El: 2410
Top of Lining El: 2405
Reinforced Concrete lining
Profile of original ground on ℄ of Spillway
Top of Excavated Slope
Duct for valve operating pipe
Spillway Crest El: 2385
S:.10
Profile on ℄ of Spillway
Spillway Ogee
Grout Holes

(c) LONGITUDINAL SECTION - SPILLWAY CREST

10'
0
10'
20'
30'

FIG. 2.—SAN GABRIEL DAM NO. 2

least dimension. This rock was to be placed at the downstream toe and on the downstream face of the dam.

Class C rock was to vary in weight from quarry chips to 14,000 pounds, the relative proportion of the various sizes to be regulated according to the requirements of placing to result in a packed rockfill of maximum density.

The source of all rock was to be a quarry one and one-half miles upstream from the dam site opened by the Flood Control District in Devils Canyon, a tributary to the West Fork of the San Gabriel River (Fig. 1). This rock had been tested and found to satisfy the above requirements. It is a granitic gneiss. Typical test results as averages of 211 tests were as follows:

Specific gravity: 2.80 or 174.7 p.c.f.
Moisture content when saturated: 0.45 per cent
Breakdown due to max. 18 feet drop: 5.04 per cent
Compressive strength: 6629 p.s.i.

Loose rockfill was to be placed in lifts of not to exceed 25 feet. It was to be spread approximately level and parallel to the axis of the dam. Placing of the fill from the abutments toward the center and thus leaving a gap at or near the center was not to be permitted. Due to scarcity of stream flow (1932), sluicing for consolidation of the loose rockfill was not provided.

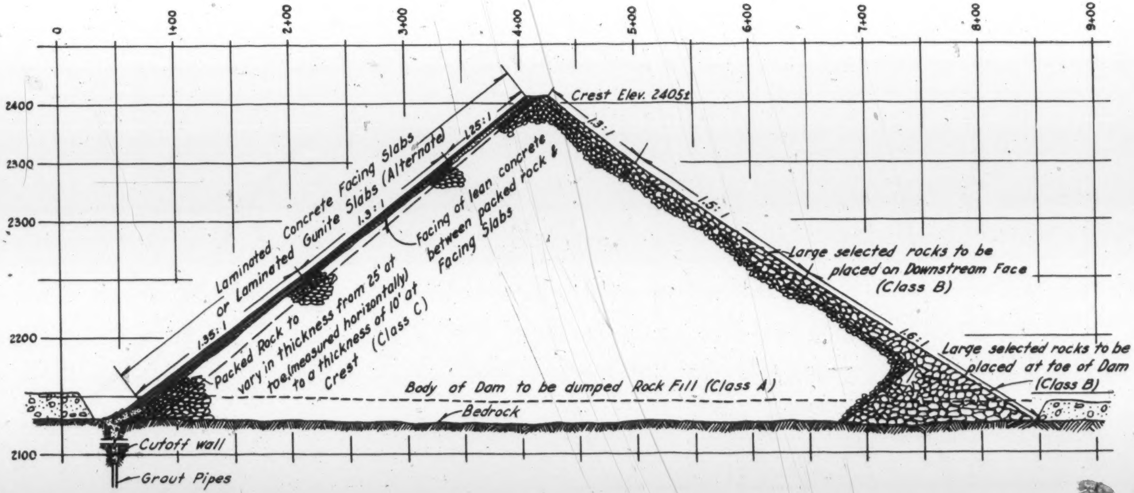
Packed rockfill was to be so placed that the natural bedding planes were to be between horizontal surfaces and surfaces approximately normal to the face of the dam. Great care was to be taken in the construction of this packed rockfill and particularly in the keying into the loose rockfill below.

Stream diversion during construction of the dam was to be accomplished through the outlet tunnel shown in the plan view of Fig. 2. Hence, this tunnel was to be constructed first.

The spillway, capable of carrying the maximum flood peak of 56,000 c.f.s. that could reasonably be anticipated, was located in a cut through the right abutment of the dam as shown in the plan view of Fig. 2a and profile and sections in Figs. 2b, c and d. A reinforced concrete arch bridge, not shown in Fig. 2a, was to provide access to the dam from the main road to the south. Capacity and area curves of the reservoir are shown in Fig. 2c.

As shown in Figs. 3a, b and c, the maximum cross section, profile along center line of crest, and projected plan of upstream face of the dam, the design followed conventional features of rockfill dams except for the facing slab which was intended to have greater flexibility than the more conventional type of facing slab. Fig. 4 shows details a, b and c of this facing slab, the distinguishing features being the lamination of the slab proper, its anchorage to the sub-slab, and attachment to the floater slab which in turn was hinged to the cutoff wall. Through this arrangement, it was hoped to provide sufficient flexibility for the slab to adjust itself to residual settlement of the dam over a period of years of its initial performance.

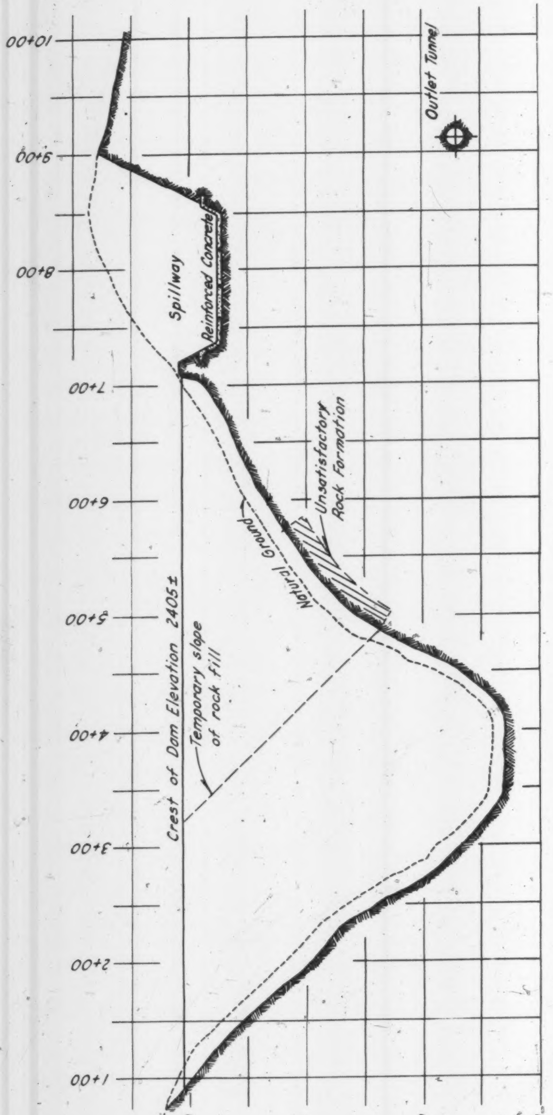
The salient provisions were as follows: A sub-slab placed directly on the packed rock in 30-foot squares with six-inch open joints between the squares; anchor bars set in the packed rock protruding through the sub-slab and serving as anchorage for the laminated facing; the number of laminations of the facing varying between five at the toe of the dam for concrete, or four for shotcrete (gunite) to two at the crest for either concrete or shotcrete; recesses in the packed rock provided at points of anchorage which were made an integral part of the sub-slab in form of spurs to prevent slippage.


0
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
2400
2300
2200
2100
Crest Elev. 2405±
Laminated Concrete Facing Slabs
or Laminated Gunite Slabs (Alternate)
1.25:1
1.3:1
1.35:1
Facing of lean concrete between packed rock & Facing Slabs
Packed Rock to vary in thickness from 25' at toe,(measured horizontally) to a thickness of 10' at Crest (Class C)
1.3:1
1.5:1
1.6:1
Large selected rocks to be placed on Downstream Face (Class B)
Large selected rocks to be placed at toe of Dam (Class B)
Body of Dam to be dumped Rock Fill (Class A)
Bedrock
Cutoff wall
Grout Pipes

(a) MAXIMUM SECTION OF DAM

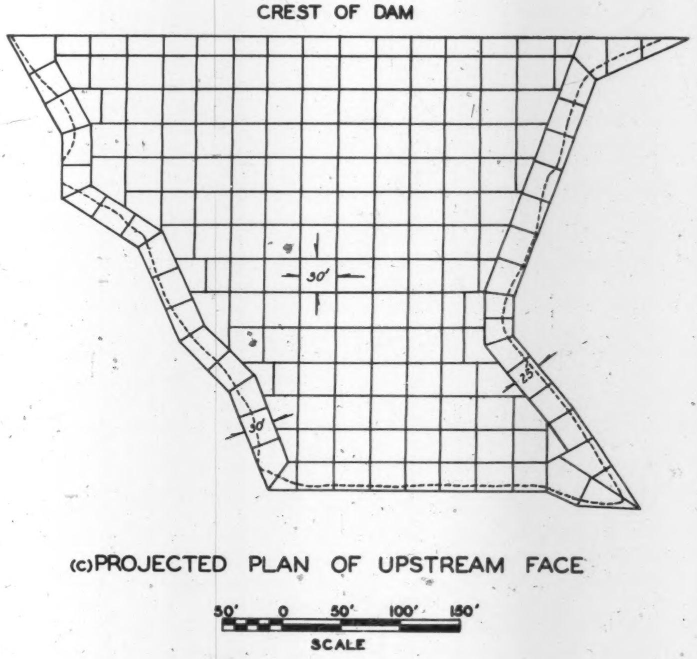
50'
0
50'
100'
150'

FIG. 3.—SAN GABRIEL DAM NO. 2


10+00
9+00
8+00
7+00
6+00
5+00
4+00
3+00
2+00
1+00
2400
2300
2200
2100
Outlet Tunnel
Spillway
Reinforced Concrete
Unsatisfactory Rock Formation
Natural Ground
Crest of Dam Elevation 2405±
Temporary slope of rock fill

(b) PROFILE ALONG ℄ OF CREST

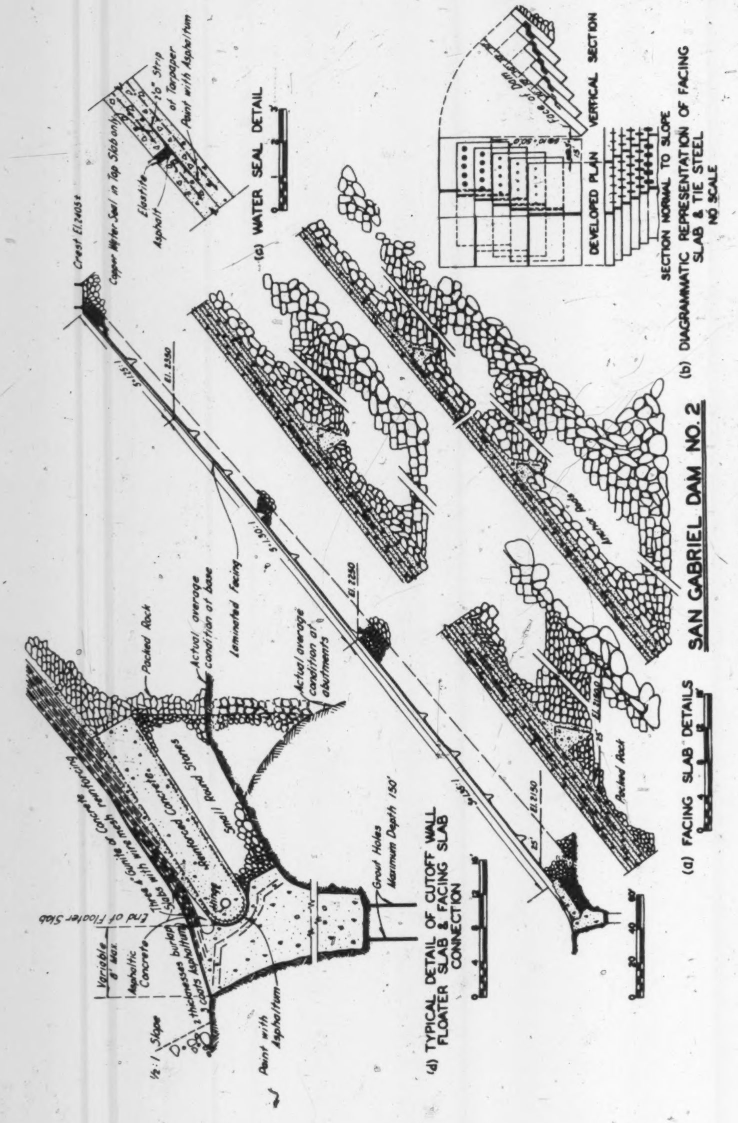
FIG. 3.—CONTINUED



(c) PROJECTED PLAN OF UPSTREAM FACE

CREST OF DAM
30'
30'
25'
50'
0
50'
100'
150'
SCALE

FIG. 3.—*CONTINUED*


SAN GABRIEL DAM NO. 2
½:1 Slope
Variable 8' Max.
Asphaltic Concrete
End of Floater Slab
2 thicknesses burlap 3 coats Asphaltum
Three 1" Gunite of Concrete reinforcing slabs with wire mesh
Reinforced Concrete
Small Round Stones
Hinge
Paint with Asphaltum
Grout Holes Maximum Depth 150'
Packed Rock
Actual average condition at base
Laminated Facing
Actual average condition at abutments
Crest El. 2405±
El. 2350
El. 2250
El. 2190
El. 2150
25°
Packed Rock
Copper Water Seal in Top Slab only
Elastite
Asphalt
2'0" Strip of Tarpaper
Paint with Asphaltum
Face of Dam
DEVELOPED PLAN
VERTICAL SECTION
SECTION NORMAL TO SLOPE

(a) FACING SLAB DETAILS

(b) DIAGRAMMATIC REPRESENTATION OF FACING SLAB & TIE STEEL
NO SCALE

(c) WATER SEAL DETAIL

(d) TYPICAL DETAIL OF CUTOFF WALL FLOATER SLAB & FACING SLAB CONNECTION

FIG. 4.—SAN GABRIEL DAM NO. 2

Perhaps the most novel feature was the floater slab hinged to the cutoff wall as shown in detail in Fig. 4d. It is obvious that for this floater slab to rotate around the hinge which was formed by short lengths of 10 in. extra heavy pipe, the rock underneath described as "small round stone" (under Class A rock) would have to yield. Furthermore, such yield would have to be proportional to the distance from the hinge so as to create uniformly distributed bearing pressure on the floater slab and thereby avoid critical bending stresses. While this requirement should reasonably be satisfied along the base of the dam, along the abutment it was problematical because of the unavoidable shattering and sloughing of the rock due to blasting in connection with benching and trenching for the construction of the cutoff wall. Hence, instead of the ideal rock section at the base as shown in Fig. 4d (solid line), the actual section at the abutments could readily turn out to conform to the one indicated by a dash line. Therefore, the small round stone fill supporting the floater slab next to the cutoff wall could act as a fulcrum due to its shallow depth as compared to the much greater depth with increasing distance toward the rock face from the cutoff wall and therefore from the hinge. For small residual settlement and deflection of the rockfill, especially as a result of full water pressure on the facing, this fulcrum effect could have been relatively harmless and the stresses in the floater slab could well have been within allowable limits. However, as a result of major settlement and deflection, the floater slab would be unable to rotate freely and to follow the deflection of the facing because of the fulcrum support. This could only lead to over-stressing and cracking at the top of the floater slab near the hinge.

As also shown in Fig. 3a, the design provided for the excavation of the stream bed materials and for the placing of the rockfill on bedrock along all of the contact surfaces of the dam. These contact surfaces were to be stripped to rock of sufficient strength to support the rockfill of the dam. The cutoff wall along the entire contact line of the face of the dam was to be carried to sound rock and was to be grouted along its entire length so as to form an effective water seal. (See Fig. 4)

The outlet works, shown in Fig. 5 were arranged in form of circular, inclined shafts varying between 7'-0" and 8'-6" net inside diameter connected to the circular outlet tunnel, 14 feet in net inside diameter, which, as previously mentioned, served as diversion tunnel during construction. Each of the four inclined shafts was to be controlled by an 84-inch butterfly valve, protected by a cage-type trash rack with 6-inch bar spacing. In addition, two 84-inch butterfly valves were provided at the upstream portal of the main tunnel likewise protected by a cage-type trash rack structure. All butterfly valves were to be hydraulically controlled from the gate house shown in Fig. 2a. (Actually the gate control house was located on the right or south side of the spillway.) Water supply of adequate pressure for the operation of the valves was provided through a storage tank above the south or right abutment at elevation 3230 with water surface at elevation 3236. Thus, the water pressure at the lowest butterfly valve was to be 459 p.s.i. For the supply of water to the tank, a pump station near the lower portal of the main tunnel was provided. Vents for all butterfly valves were likewise provided (Fig. 2) through pipes anchored to the cliff of the right abutment with the air intake structure at elev. 2408.5 or 10.5 feet above highest water level of the reservoir.

To check qualitatively the complex hydraulic computations attending the outlet works, model tests were performed. The model, was made of transparent, plastic material which permitted close observation of possible

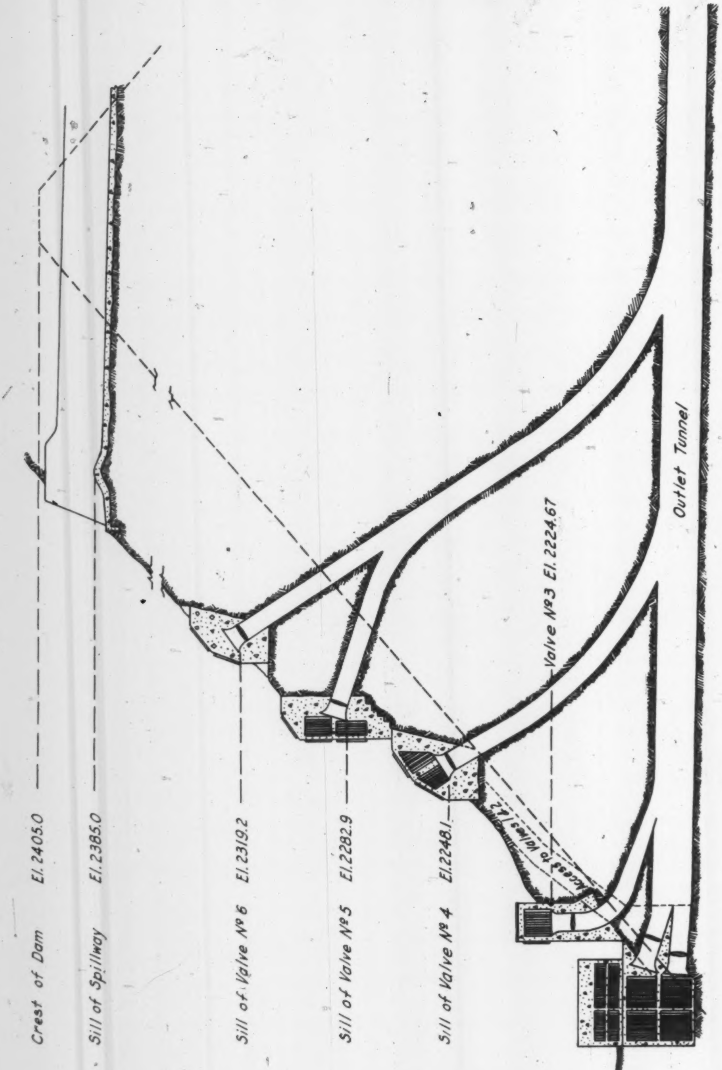

Crest of Dam El. 2405.0
Sill of Spillway El. 2385.0
Sill of Valve Nº 6 El. 2319.2
Sill of Valve Nº 5 El. 2282.9
Sill of Valve Nº 4 El. 2248.1
Valve Nº 3 El. 2224.67
Access to Valves 1 & 2
Outlet Tunnel

FIG. 5.—OUTLET WORKS OF SAN GABRIEL DAM NO. 2

cavitation or other undesirable features under various operating conditions.

Construction

Construction of Cogswell Dam started in March, 1932 and progressed according to schedule until the Spring of 1933. At this time, difficulties were encountered in the finding of satisfactory rock at which to terminate the cutoff wall approximately between elevs. 2235 and 2320 of the south or right abutment. This distance is indicated in Fig. 3b. Further placing of rockfill against this reach of the abutment was halted by the California State Engineer pending additional deepening of the cutoff trench in quest of satisfactory rock. Meanwhile, however, the placing of rockfill continued along the left abutment, leading to a slope, corresponding to the angle of repose of the dry rock as indicated by a dash line in Fig. 3b. Hence, the very condition was thereby created which, according to the specifications, was to be avoided.

After further deepening of the cutoff trench did not reveal satisfactory rock and the fill at and adjacent to the north or left abutment had reached the crest of the dam, it was decided to proceed with the pouring of the cutoff wall in the reach in question and to subsequently complete its base through tunneling and stoping. This arrangement made it possible to fill the V-shaped gap by dumping the rock from the completed level, thus initially from a much greater lift than the maximum 25 feet provided in the specifications. Under these conditions, arching between the fill slope and the abutment wall was unavoidable.

The contractor chose the use of shotcrete (gunite) as an alternate to concrete for the sub-slab and the laminated facing as well as for the lining of the inclined shafts of the outlet works. Construction of the facing followed the placing of the rockfill and the packed rock part thereof, with a minimum of necessary lag and by December of 1933 had been about 80 per cent completed.

On December 31, a major storm swept in from the Pacific Ocean which by noon of January 1, 1934 had yielded 15.07 inches of rain at the dam. The application of this natural lubrication to the dry rockfill caused the latter to settle, especially that part dumped in the gap at the right abutment. Immediate settlement there amounted to some 4 per cent. Vertical settlement was accompanied by slight bulging in the upstream lower half of the dam, resulting in damage to the laminated facing and the sub-slab, particularly near the abutments. The packed rock section which in effect was an inclined, dry rock wall resting on the dumped rockfill below, could not follow the settlement of the latter because of its greater rigidity. Consequently, the packed rock section buckled, thereby adding to the destruction of the facing to that which would have occurred had the settlement been entirely plastic.

Pending appraisal of the damage and preparation of remedial measures, the contract work came to a standstill.

The storm reversed the water supply from deficient to abundant. Thus, water in ample volume was now available for artificial sluicing of the rockfill. This was accomplished by drilling holes through facing and sub-slab, setting pipe with fire hose connectors into these holes, and pumping clear water into the rockfill to hasten final settlement. After several months of this sluicing operation, the afore-mentioned, maximum settlement of 4 per cent had been increased to 6 per cent. The average residual settlement amounted to 4.5 per cent. A nearly stable condition was indicated so far as the influence of dead weight was concerned. However, plastic deformation under full water load still had to be anticipated.

Separate Contract Work

The replacement of the damaged facing by a temporary timber facing, as shown in Fig. 6, was undertaken by separate contract. This work included the removal of the laminated facing, the repair of the sub-slab and the placing of the timber facing. Also under separate contract additional rock was derrick-placed on the crest of the dam. Both contracts were completed in the Summer of 1935. As may be seen from details a, b, c and d of Fig. 6, the timber facing was assembled from sections, placed horizontally, of three 2'' x 10'' x 10' minimum, random length planks of untreated douglas fir, nailed together to provide tongue and groove interlocks on all four sides, similar to Wakefield sheet piling. Roofing nails along the top groove of each section provided an initial clearance of 1/4 in. in anticipation of additional settlement and strips of galvanized iron were provided to cover open joints at ends of three-plank sections. Walers in vertical planes anchored to the rockfill below served to hold the facing in place and to avoid floatation. It was expected that all but negligible settlements would cease to exist within five years from the time of completion of the timber facing, that the latter would last that long and would then be replaced by a permanent, reinforced concrete facing of more nearly conventional design. Actually, the timber facing lasted more than twice the anticipated period of its lifetime.

Performance Under Full Water Load

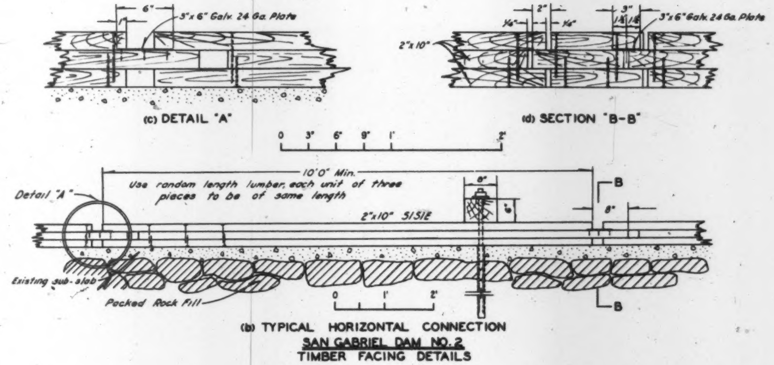
An intense storm between February 26 and 28, 1938, which was followed by a major storm between March 2 and March 5, produced the largest flood of record in Los Angeles County. All the flood peaks occurred on March 2. At Cogswell Dam the peak inflow amounted to nearly 25,000 c.f.s., the reservoir filled in 71 hours and the flow over the spillway peaked at 23,400 c.f.s., although the outlets, with the exception of the lowest two, discharging directly into the main tunnel, remained operative. The lowest two outlets became inoperative after the cage type trash rack collapsed because of clogging by water-logged trash.

The leakage through the timber facing and the dam reached a maximum of 130 c.f.s., whereas, calculations, based on uniform porosity, indicated that leakage for full water load might reach 200 c.f.s. which would still have been less than half the rate of leakage that might prove to be critical. Due to the swelling of the timber facing the leakage was smaller for full than for partial water load.

Damage to Floater Slab

While the timber facing suffered no structural damage under full water load, the floater slab at both abutments developed a crack, extending from the base of the dam to roughly one-half of its height. This was due to bending stresses caused by the afore-mentioned fulcrum effect. The reinforcing steel near the top of the slab was either stressed beyond its elastic limit or had broken the bond by shearing off in form of a spall, the concrete cover. In either case the crack was of such width as to contribute materially to the leakage through the dam. Fig. 7 shows the crack in question near the base of the left abutment.

(a) Placing



(c) DETAIL "A"

(d) SECTION "B-B"

(b) TYPICAL HORIZONTAL CONNECTION

SAN GABRIEL DAM NO. 2
TIMBER FACING DETAILS

6"
1"
3"x 6" Galv. 24 Ga. Plate
2"
1/8"
1/8"
3"
1 1/2
1 1/2
3"x 6" Galv. 24 Ga. Plate
2"x 10"
0 3" 6" 9" 1' 2'
10'0" Min.
Use random length lumber, each unit of three pieces to be of same length
Detail "A"
8"
B
8"
2"x10" S1S1E
Existing sub-slab
Packed Rock Fill
0 1' 2'
B

FIG. 6.—TIMBER FACING DETAILS OF SAN GABRIEL DAM NO. 2

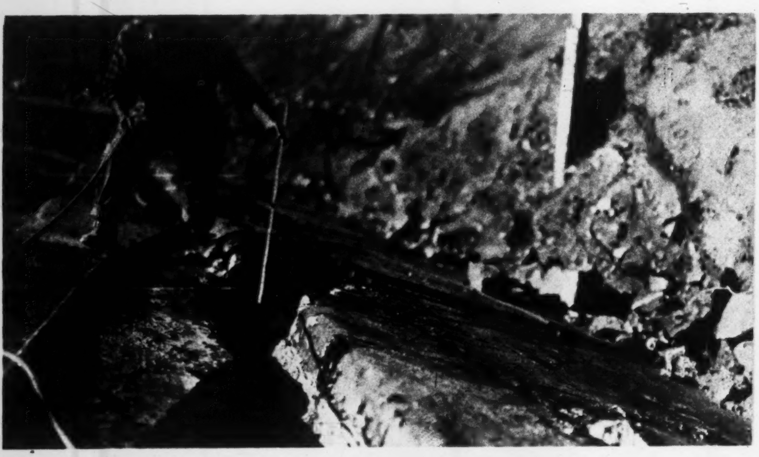


FIG. 7.—FLOATER SLAB FAILURE NEAR BASE OF LEFT ABUTMENT OF SAN GABRIEL DAM NO. 2

FIG. 8.—PARTIALLY COMPLETED TOWER TYPE TRASH RACK FOR SAN GABRIEL DAM NO. 2

Revision of Trash Rack

The collapse of the main trash rack called for the change from the cage type to the riser or tower type of trash rack, partial completion of which is shown in Fig. 8. This was accomplished in 1939 by District forces. The reinforced concrete structure was designed to support 340 feet of water column at the base, whereas the trash rack bars were designed for 224 feet of water column, although the design of the trash rack was such as to render clogging extremely improbable. As shown in Fig. 9 the bars were fabricated, tapered box sections with a clear spacing of 15.0 in. on the outside and 15.2 in. on the inside. Hence, any piece of trash clearing the outside spacing will move through without danger of wedging between the bars.

During the flood which culminated on January 23, 1943 the tower type trash rack performed to full satisfaction, although the trash produced was quite comparable in volume with that of the March 2, 1938 flood.

Control of Outlets

The design of the valves for the control of the outlets followed the important requirement for flood control operation of relative invulnerability to clogging by trash having passed between the trash rack bars. Accordingly, as a "rule of thumb," the minimum valve opening should be about twice the maximum trash bar spacing. This rule was satisfied at all outlets including the two 84-inch butterfly valves, protected by the tower trash rack. There is, of course, no harm done if the trash rack spacing is less than one-half of the minimum valve opening, so long as the trash racks are properly designed for water pressure in light of the increasing likelihood of clogging with diminishing bar spacing. However, for reasons of economy in flood operation, it is desirable to have as much trash as possible carried through the outlets by the water and thereby save the expense of manual removal. Obviously, trash such as logs that could cause serious damage to structures or facilities below a flood control dam, should not be permitted to pass through the outlets. Exempt from this restriction naturally is uncontrolled spillway flow.

The control of the outlets at Cogswell Dam, while advantageous for flood operation proved less desirable for operation in connection with water conservation. This was because of the fact that butterfly valves are not stable for part opening, that they should be either fully closed or fully open, and that when fully open the discharge may be far in excess of that desired. To make release of small flows possible, the lowest of the inclined shafts was converted in 1953 by District forces, to an outlet controlled by a 16-inch hollow jet valve.

Permanent Reinforced Concrete Facing

The design of the permanent, reinforced concrete facing, constructed in 1947 under contract, is evident from details shown in Fig. 10. To preserve the integrity of the facing in case of additional settlement, possibly due to a combination of full water load and earthquake forces, special attention was paid to the copper water seal, particularly at junction points of three and four seal sections. This was solved by brazing these sections to a semispheric shell which could stand considerable distortion without rupture. The facing was again articulated in 30' x 30' square units except near the abutments where the shape of the units had to be adapted to the respective

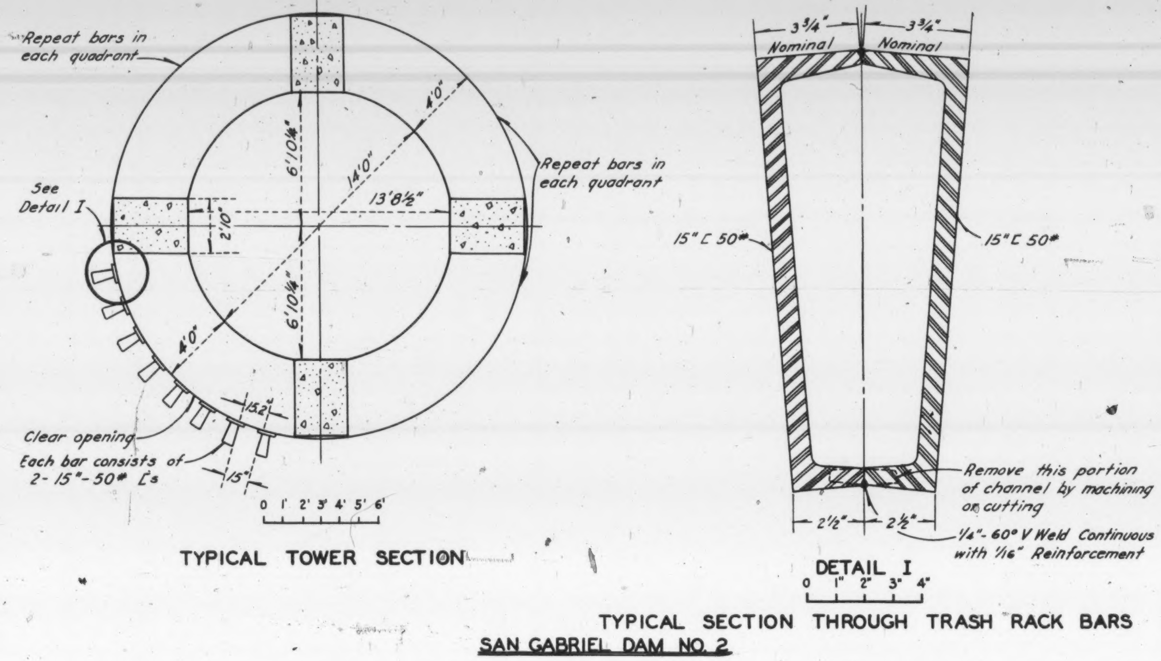

Repeat bars in each quadrant
See Detail I
4'0"
6'10¼"
14'0"
13'8½"
2'0"
6'10¼"
4'0"
Repeat bars in each quadrant
15.2"
Clear opening
Each bar consists of 2- 15"-50# [s
15"
0 1' 2' 3' 4' 5' 6'
TYPICAL TOWER SECTION
3¾" Nominal
3¾" Nominal
15" [50#
15" [50#
Remove this portion of channel by machining or cutting
2½"
2½"
¼"-60° V Weld Continuous with 1/16" Reinforcement
DETAIL I
0 1" 2" 3" 4"
TYPICAL SECTION THROUGH TRASH RACK BARS
SAN GABRIEL DAM NO. 2

FIG. 9.—TRASH RACK TOWER DETAILS FOR SAN GABRIEL DAM NO. 2

configuration. The reinforced concrete facing was terminated along the 2375 contour and the portion above constructed of broom-grouted rock by District forces in 1948.

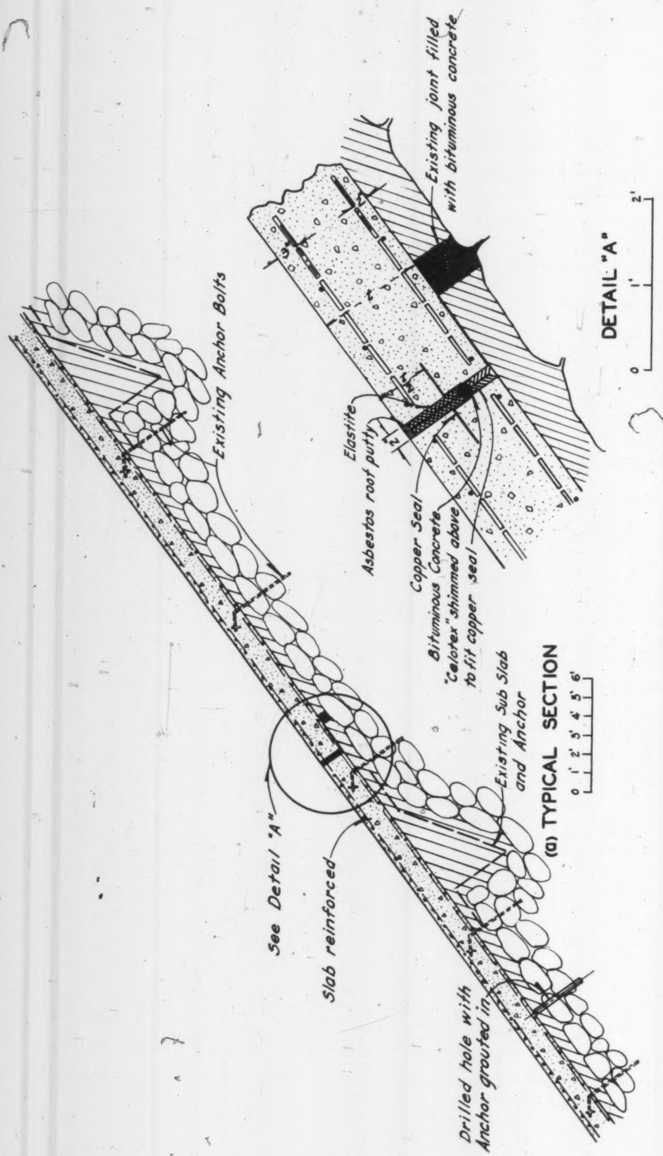
Initial and Residual Settlement

A brief review of the shrinkage and settlement of the rockfill, during and after construction, should prove of interest to the profession and possibly call for caution so far as rockfill dams of major dimensions are concerned. The contractor chose to haul the rock from the quarry to the dam by a fleet of trucks of nominal 10 cu. yds. capacity, capable of carrying 12 cu. yds. The average volume per truck amounted to 11.5 cu. yds. Exact count of trucks delivering rock to the dam was kept and the rockfill measured after placing. Thus, it was possible to derive the relation between truck count and yield in volume of fill in the dam at various elevations. The result is shown in Fig. 11; namely, that the yield of each truck load was 11.3 cu. yds., for the first 25-foot lift and only 9.4 cu. yds. by the time the dam had reached its maximum height. This corresponds to a shrinkage or consolidation during construction of roughly 18 per cent. Residual settlement of 6 per cent previously mentioned, raised this figure to a maximum of 24 per cent and an average of 22.5 per cent.

Significance of Settlement

Loose rock on the truck, weighing 105 p.c.f. (based on 174.7 p.c.f. solid weight and 40 per cent voids exclusive of quarry dust) had consolidated to weigh 128.0 p.c.f. average in the dam during construction and to 135.5 p.c.f. subsequent thereto. Bulging in the lower half of the upstream face, previously mentioned, was too small to have had an appreciable effect on the latter figure. Hence, void space had decreased from 40 to 26.7 and 22.5 per cent., respectively. The shrinkage during construction was primarily due to the crushing of contact points, chips and small rocks with resultant swelling of the crushed material which could only find room in the voids. While this transfer of materials to the voids obviously did not change the total weight, it increased the average unit weight by decreasing the size of the voids. This situation was accentuated through the residual, that is post-construction, settlement which could only take place through additional compression of the crushed material in the voids once point contact had been eliminated. Indeed, the transmission of pressure through the compacted material in the voids is the pre-requisite for final stability of a rockfill as further crushing of contacts is thereby neutralized. To visualize rockfill dams of major height as a mass of large rock with large interstices throughout is therefore a myth.

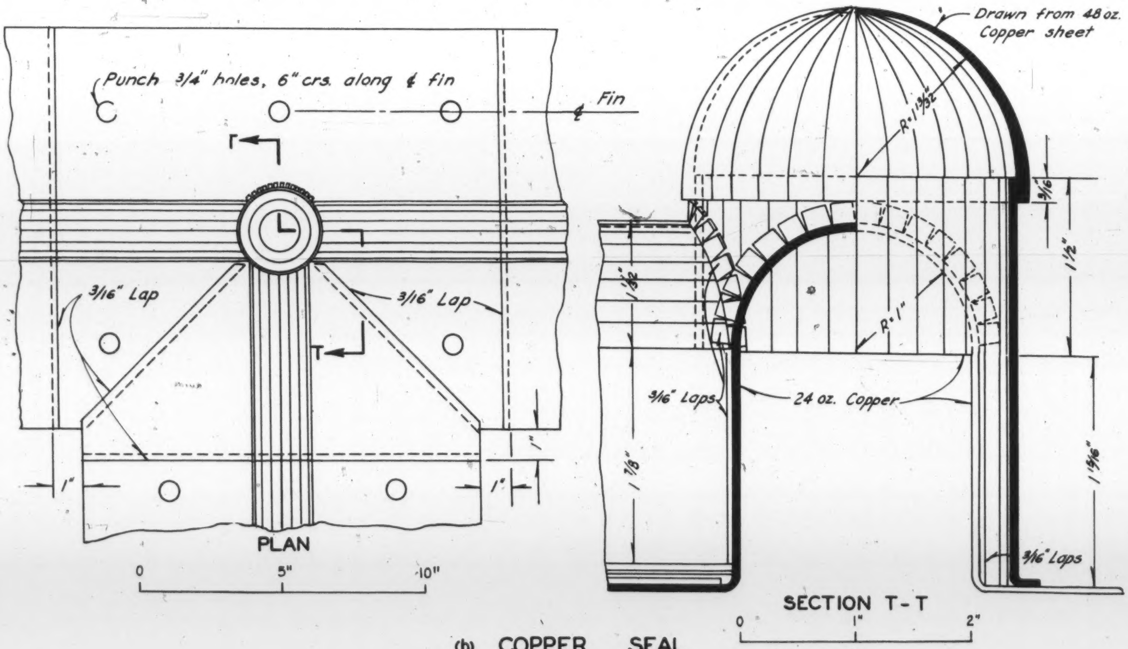
The permeability of the fill, particularly in the lower part of the dam was necessarily reduced, since consolidation as reflected in above figures is not uniform but varies from a maximum at the base to a minimum at the crest of the dam. Had, for example, the total specific consolidation been close to twice the average at the base and close to zero at the top, the void space at the base capable of transmitting significant percolation would have completely disappeared. On this basis the composite dry density at the base of the dam would be of the order of 156 p.c.f. and the void space 11 per cent. The 105 lbs. of solid material which at 40 per cent voids had occupied one cubic foot of volume prior to consolidation during and after construction would now occupy .67 cu. ft., more or less. It now remains to be ascertained under what



(a) TYPICAL SECTION

DETAIL "A"

Existing Anchor Bolts
See Detail "A"
Slab reinforced
Drilled hole with Anchor grouted in
Existing Sub Slab and Anchor
0 1' 2' 3' 4' 5' 6'
Existing joint filled with bituminous concrete
Elastite
Asbestos roof putty
Copper Seal
Bituminous Concrete
"Celotex" shimmed above to fit copper seal
3"
2"
t
0 1' 2'


Punch 3/4" holes, 6" crs. along ℄ fin
℄ Fin
T
T
3/16" Lap
3/16" Lap
1"
1"
1"
PLAN
0
5"
10"
Drawn from 48 oz. Copper sheet
R=1 13/32"
9/16
1½"
1 1/32"
R=1"
3/16" Laps
24 oz. Copper
1 7/8"
1 9/16"
3/16" Laps
SECTION T-T
0
1"
2"

(b) COPPER SEAL

DETAIL OF TEE
PREFABRICATED

FIG. 10.—ALTERATIONS TO REINFORCED CONCRETE FACING SLAB OF SAN GABRIEL DAM NO. 2

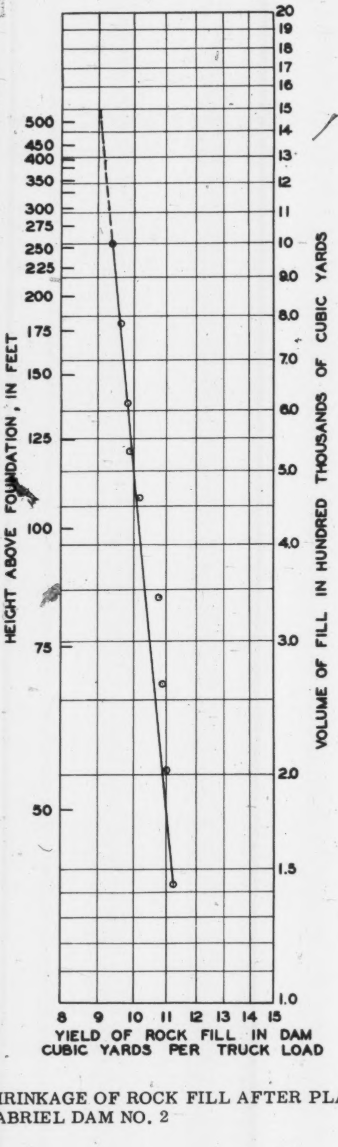


FIG. 11.—SHRINKAGE OF ROCK FILL AFTER PLACING IN SAN GABRIEL DAM NO. 2

conditions the crushed materials, together with the quarry dust could (1) fill the voids, and (2) be compressed to such a density as would render this combination of materials practically impervious.

The weight of the quarry dust which has so far been neglected, must now be added to the weight of the crushed materials. The specifications allowed 3 per cent of the total weight of Class A rock in quarry dust. Hence, quarry dust weighed 3.2 pounds. If in the course of shrinkage during construction 75 per cent of the original 105 lbs. of rock remained uncrushed and 25 per cent of crushed material and quarry dust filled the voids, then the dry density of the combined materials filling the voids would be 118 p.c.f. Thus, in accordance with this premise which seems to be reasonable, the materials filling the void space at the base of the dam were already approaching impermeability prior to post-construction settlement. The corresponding composite dry density, including quarry dust, would be 154.5 p.c.f.

Consolidation due to the residual settlement raised the dry density of crushed materials and quarry dust to 132 p.c.f. or, to at least 95 per cent relative density based on 40,000 ft. lbs of compactive effort. The corresponding composite dry density, including quarry dust, would be 159.2 p.c.f. Hence, even if specific consolidation at the base had been less than twice the average for the entire dam, the rockfill there would still be far from free draining.

Therefore, rockfill dams of major height can by no means be considered immune to pore pressure, especially in the interior of the lower part of the dam where densities would normally be greatest. In brief, this means that a core of low permeability is likely to be formed in such dams whether it be intended or not.

Fig. 12 shows the dam with the permanent facing completed.

Performance of San Gabriel Dam

Original Design

In view of the previously mentioned paper on the "Design and Construction of San Gabriel Dam No. 1" the performance thereof will be confined to those features which are contra-distinct in relation to Cogswell Dam and to those not included in the afore-mentioned paper. Actually the paper refers to the revision of the dam as originally proposed. Basically, the original design was identical with that of Cogswell Dam. However, certain adaptations to the additional hundred feet in height were necessary, among them the number of face laminations and the upstream and downstream slopes. Had the quality of rock, method of placing, and shape of dam site been identical with those at Cogswell Dam the shrinkage during construction as an average for the entire section would, in accordance with Fig. 11, have been 21 per cent and the average total shrinkage 25.5 per cent. Consequently, in view of above statement regarding non uniform distribution of densities at least the lowest 100 feet of the interior of San Gabriel Dam, as originally planned, would have been virtually impervious. Yet, the concept (not necessarily confined to the subject dams) has been that any leakage through the facing would be carried away through the mass of free draining rock and that uplift on the facing in case of rapid draw-down of the reservoir could not occur. With rockfill dams of moderate height and high quality rock relatively free drainage may exist, whereas the assumption that some channels would always be open for drainage,

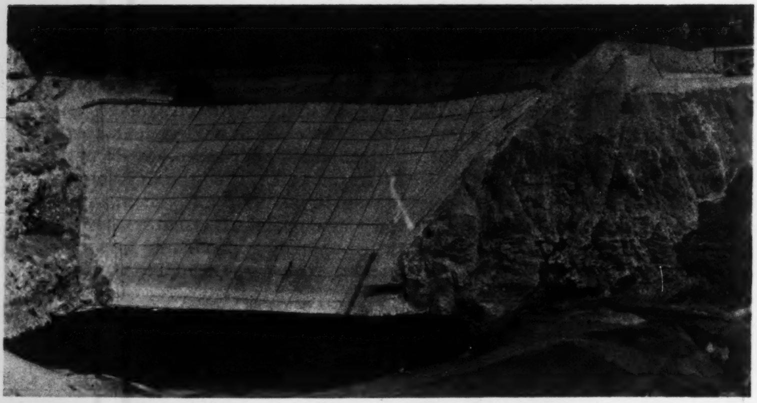


FIG. 12.—COMPLETED PERMANENT FACING FOR SAN GABRIEL DAM NO. 2

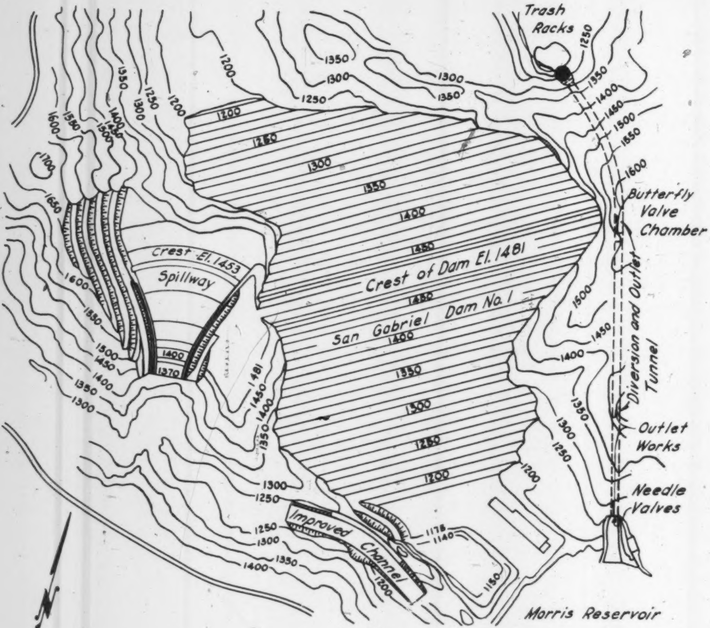

Trash Racks
Butterfly Valve Chamber
Crest El. 1453
Spillway
Crest of Dam El. 1481
San Gabriel Dam No. 1
Diversion and Outlet Tunnel
Outlet Works
Needle Valves
Improved Channel
Morris Reservoir

FIG. 13.—PLAN VIEW OF SAN GABRIEL DAM NO. 1

regardless of height, is purely speculative and has no justification in fact. Actually, the rock quarried immediately below San Gabriel Dam (Quarry No. 10) was far inferior in quality to that at Cogswell Dam and proved to be unsuitable for the type of structure proposed. The break-down was such that the fines predominated. It therefore became necessary to revise the design so as to incorporate the fines in the mass of the dam. Fines are defined as particles 1/4 in. in size and smaller.

Revised Design

Figs. 13 and 14 show the revised design of San Gabriel Dam in plan and section. Conspicuous above all, are the flattened faces and the division of the section into six zones. Among the latter zone 3, consisting of quarry rock which had passed through 6'' x 9'' "grizzly" openings, corresponded approximately in cross section, to the original loose rockfill section. The material in this zone, with rock up to 9 in. in size and fines graded down to quarry dust averaging some 60 per cent by volume, was mechanically compacted to a minimum dry density of the fines of 120 p.c.f. Actually, this density averaged 122.2 p.c.f. for zone 3. The composite dry density (rock and fines) averaged 144 p.c.f. Hence, in principle, this quarry material was artificially compacted to a similar density that existed near the base of Cogswell Dam due to self compaction. The zone 3 material was relatively impervious, namely, it showed a percolation rate of 3 feet per year under a gradient of 1 to 1. Had it not been for the earthquake hazard in this part of the country, zone 2, a blanket of moderately compacted clayey sand or loam, would have been superfluous. However, under the circumstances, this blanket, treated so as to remain plastic, was intended to seal possible rupture of zone 3 caused by earthquake.

Rock of best available quality was placed in zones 5 and 6, and on the face of zone 1. Otherwise, the rockfill was a mixture of coarse and fine material, roughly graded from finest next to zones 2 and 3 to coarsest toward the face rock of zone 1 and zones 5 and 6.

All loose rock was placed by the use of 2 cu. yds. of water to 1 cu. yd. of rock. Initial settlement, thereby produced averaged 6 per cent. This checks the afore-mentioned settlement at Cogswell Dam due to rain and sluicing after dry placing of the rock.

Great care was exercised in the construction of the cutoff wall which varied in depth between 25 and 80 feet, more or less. Depth of grout holes averaged an additional 73 feet.

The design of the spillway, placed in a cut through the right or west abutment was similar to that at Cogswell Dam. Its maximum capacity is 290,000 c.f.s. for the controlled and uncontrolled drainage area of 202.7 square miles.

The diversion tunnel, 30 feet in net diameter, served to accommodate the outlet works shown in Fig. 15. Before completion thereof, the aforementioned flood of March 2, 1938 that produced a peak inflow of nearly 100,000 c.f.s. on March 2, 1938 and a spillway discharge peak of 56,700 c.f.s., filled the reservoir in 73 hours. It deposited over ten million cubic yards of debris reaching some 60 feet in depth at the outlet tower. To preserve the effectiveness of the latter, two tiers of trash rack were added in the Summer of 1939. The dam proper was completed in the Summer of 1937, the spillway two weeks before the referenced flood, and the outlet works in 1939. Under the maximum water load the dam deflected downstream 0.11 feet at the crest and returned

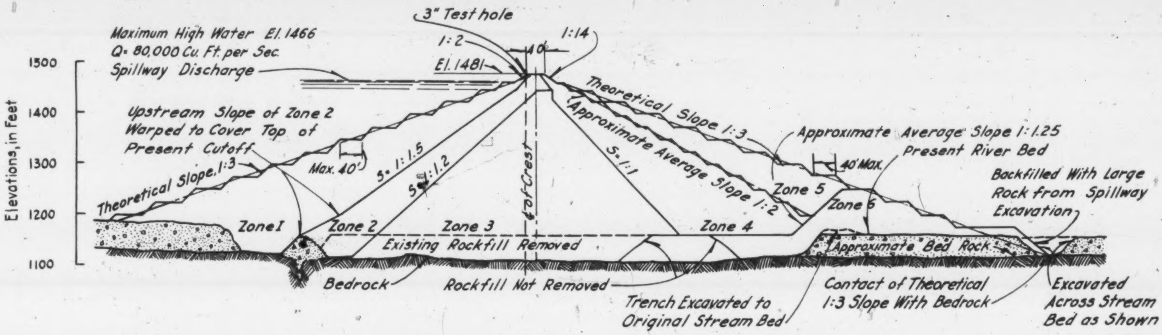

Elevations, in Feet
1500
1400
1300
1200
1100
Maximum High Water El. 1466
Q= 80,000 Cu. Ft. per Sec.
Spillway Discharge
3" Test hole
1:2
El. 1481
10
1:14
Upstream Slope of Zone 2
Warped to Cover Top of
Present Cutoff
Theoretical Slope, 1:3
Max. 40'
S= 1:1.5
S= 1:1.2
℄ of Crest
S= 1:1
Theoretical Slope 1:3
Approximate Average Slope 1:2
Approximate Average Slope 1:1.25
Present River Bed
40' Max.
Zone 1
Zone 2
Zone 3
Zone 4
Zone 5
Zone 6
Existing Rockfill Removed
Approximate Bed Rock
Backfilled With Large
Rock from Spillway
Excavation
Bedrock
Rockfill Not Removed
Trench Excavated to
Original Stream Bed
Contact of Theoretical
1:3 Slope With Bedrock
Excavated
Across Stream
Bed as Shown

FIG. 14.—COMPOSITE MAXIMUM SECTION, SAN GABRIEL DAM NO. 1

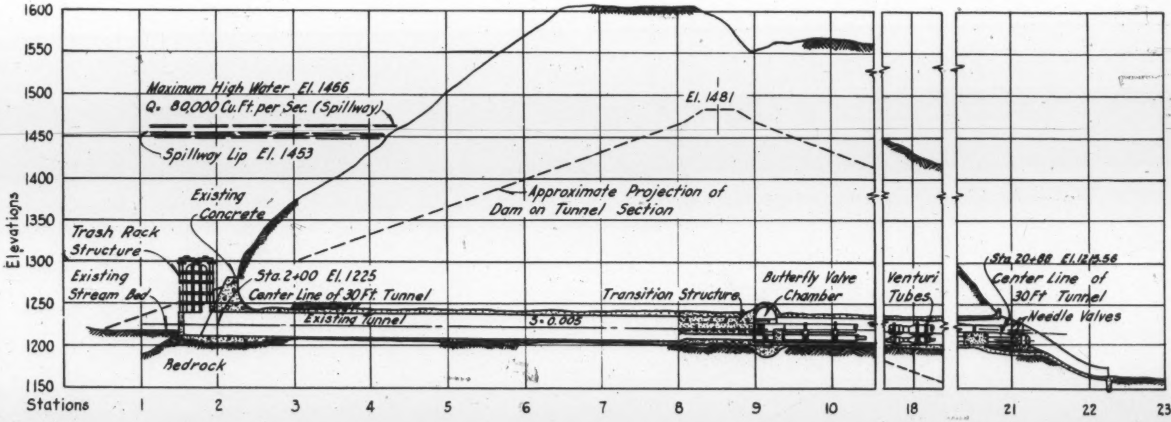

1600
1550
1500
1450
1400
1350
1300
1250
1200
1150
Elevations
Stations
1
2
3
4
5
6
7
8
9
10
18
21
22
23
Maximum High Water El. 1466
Q. 80,000 Cu. Ft. per Sec. (Spillway)
Spillway Lip El. 1453
El. 1481
Approximate Projection of Dam on Tunnel Section
Existing Concrete
Trash Rack Structure
Existing Stream Bed
Sta. 2+00 El. 1225
Center Line of 30 Ft. Tunnel
Existing Tunnel
Bedrock
S = 0.005
Transition Structure
Butterfly Valve Chamber
Venturi Tubes
Sta. 20+88 El. 1213.56
Center Line of 30 Ft Tunnel
Needle Valves

FIG. 15.—LONGITUDINAL SECTION ON CENTER LINE OF 30 FT TUNNEL FOR SAN GABRIEL DAM NO. 1

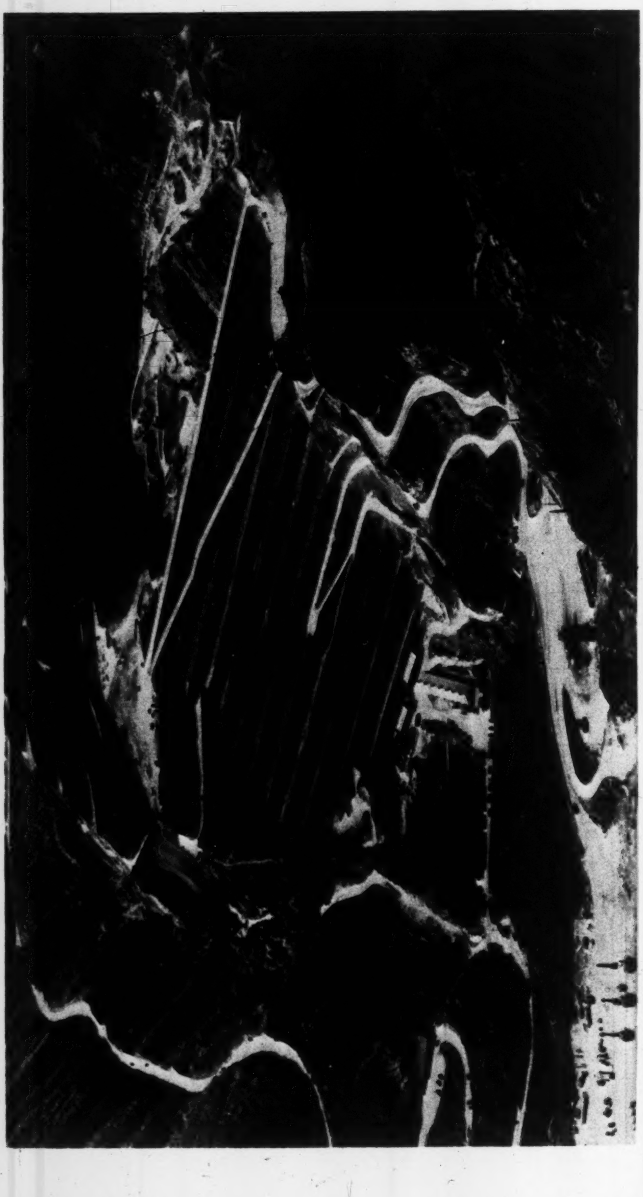


FIG. 16.—SAN GABRIEL DAM NO. 1 COMPLETED. SPILLWAY UNDER CONSTRUCTION

(elastically) within a fraction of an inch to its position before the flood when the reservoir was emptied. Maximum residual settlement at the crest to date amounts to 0.40 feet, or .106 per cent of the maximum height.

To explore the condition of zone 3 and simultaneously ascertain the position of the phreatic line, a three inch diameter hole was drilled by District forces in 1950 and lined with a perforated casing. The location is indicated in Fig. 14. It was possible to recover cored samples. The average dry density of these samples was 133.4 p.c.f., and the average moisture content 7.6 per cent.

The total residual settlement up to date of the loose rockfill, measured vertically above the downstream toe of zone 3 amounts of 1.57 feet. Fig. 16 shows the completed dam and the spillway under construction.

Epilogue

The design of a loose rockfill dam by far exceeds in simplicity the analysis of its inner stability. In fact, an exact stress analysis is impossible because of the singular absence of homogeneity and isotropy. Hence, this type of dam must be classed as empirical rather than scientific. Yet, the outer and inner stability of a properly designed rockfill dam under static loading cannot be questioned, nor the stability under both static and dynamic loading once the rockfill has fully settled. However, to prevent movement of face rock under dynamic loading, such as caused by earthquake, the slope of the faces should be flatter than that corresponding to the angle of repose. Therefore, it should at least be of the order of 1 on 1-1/2. Even a relatively small differential will, however, enhance safety against movement.

To the writer's knowledge, none of the major, "western type" rockfill dams has so far been exposed to a major earthquake. If so exposed, some settlement in addition to that previously observed would certainly have to be expected, especially in the upper part of the dam least consolidated through self-compaction. It is precisely therein that there rests the difference between a loose rockfill dam and a compacted rockfill or earth-fill dam. In the latter case, homogeneity and isotropy are mechanically created throughout the section and independent of self-compaction. Furthermore, the face slopes of compacted fill dams are invariably a multiple of the slope of repose. Where there is a choice and costs compare favorably, compacted fill dams should be given preference, especially in areas known to be subject to earthquakes.

ACKNOWLEDGMENT

The writer is indebted to Mr. H. E. Hedger, Chief Engineer of Los Angeles County Flood Control District, for his support in the preparation of this paper. To Messrs. R. D. Reeve, E. C. Kenyon, Jr. and C. J. Reinhard and Mrs. Mary K. Imandt for assembling and drawing the exhibits. Also to Mrs. Hazel M. Robertson for recording, typing and assembling the paper.

DISCUSSION

W. A. BROWN,[4] **M. ASCE.**--Mr. Baumann has seriously questioned the commonly accepted belief that large dumped rockfill dams are highly pervious. Unfortunately, a physical demonstration of impermeability within the body of a high rockfill dam is lacking and there will be much reluctance by engineers to accept this premise without further proof.

The leakage at Cogswell Dam was reported to have reached a maximum of 130 c.f.s. through the timber facing. It would ordinarily be presumed that this leakage flowed through the interstices of the rock at the base of the dam. Mr. Baumann's analysis would suggest a relatively high zone of saturation within the center of the dam and accompanying pore pressures while such leakage was occurring. It follows that the downstream slope stability would have been reduced.

As higher rockfill dams are constructed, prudent design should dictate an investigation of the degree of self-compaction in existing high rockfill dams and the resulting permeability. Such investigations might include borings and piezometer installations in existing structures, plus largescale laboratory compression tests with correspondingly large loadings on quarry rock samples.

San Gabriel Dam No. 1 has a large central core of low permeability. Because of the flat core slopes, the dumped rockfill shells serve as weighting zones. In this case, consolidation of the shells has little or no effect on the stability of the dam. Even if self-compaction reduced the permeability of the rockfill shells, they would unquestionably be much more pervious than the core. In the writer's opinion, this design is as nearly unquestionably safe as any dam can be.

It is desired to compare this design to one wherein the central core width has been reduced to a maximum of, say 10 or 20 feet, at the crest and having side slopes of 0.2 horizontally to 1.0 vertically, or steeper.

Transition zones between the core and rockfill shells immediately assume great importance to prevent migration of the fines and piping into the dumped rockfill shells.

Vertical post-construction settlement of the rockfill shells may appear to be of no more importance than in the case of the San Gabriel No. 1 type of design and of much less importance than in the Cogswell design. The formation of longitudinal cracks on the crest of such dams has frequently been observed. It is ascribed to differential settlement of the rockfill relative to the core and a slight separation at the interface. Such cracks are generally considered harmless. They do, however, indicate local lack of lateral support for the core.

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[4] Superv., Safety of Dams, California Dept. of Water Resources, Sacramento, Calif.

The thin central core, by itself, has little stability. It is essentially a membrane which is solely dependent upon the passive pressure of the downstream rock shell for stability. Conventional design methods recognize that this passive pressure must be sufficient to resist the reservoir pressure and the active pressure of the upstream rockfill shell. A sliding wedge analysis indicates that the thinner the central core is, the steeper the downstream rockfill shell slope may be. If a high value of coefficient of internal friction for the rockfill is adopted, it can even be shown that the downstream slope may be as steep as the angle of repose of the rockfill, neglecting allowances for seismic loads. This design approach should not be taken, however, without consideration of deformation.

Mr. Baumann has shown that a dumped rockfill achieves consolidation through its own weight. Crushing of the smaller rock particles and compaction of the material in the rock voids takes place due to weight. An equal degree of intimate contact and crushing is not developed in a horizontal direction. Yet with the vertical core, the water load is applied horizontally instead of toward the foundation. It is reasonable to expect, therefore, that the water load deflection will be larger in proportion to the applied load than in any other type of rockfill dam. Further, lateral deflection of an insufficiently large rockfill shell due to horizontal water load can cause failure of the core by rupturing long before a shear failure will occur in the rockfill shell.

The records of a few of the older dams in California, which had vertical concrete core walls and dumped rockfill shells, verify the above expectations. One in particular, having a maximum height of about 140 feet, deflected slowly downstream over a period of years, a maximum of nearly 13 feet at the crest. The movement was finally arrested by dumping a large volume of additional rockfill on the downstream shell to increase its mass. The shell slope was on the angle of repose of the rockfill. Fortunately, the reinforced core wall was able to undergo the deflection without complete failure in this instance.

The most adverse characteristic of a loose rockfill is its capability to deform non-elastically—from its own weight or from applied load. Post construction consolidation can be minimized by appropriate placing methods and sluicing, but neither the magnitude of this nor that of water load deformation can be pre-determined with any mathematical certainty. Dams of the Cogswell type may be expected to approach a limit of deformation within a reasonable time, and failure to provide a design which will accommodate that maximum deformation usually only means excessive leakage and repairs to the upstream facing. It is suggested, however, that much more serious consequences may be the result with the central core type of rockfill dam if too much reliance is placed upon the downstream shell as a structural supporting member rather than as a weighting zone as in the case of San Gabriel Dam No. 1.

This only emphasizes Mr. Baumann's comment that the design of a loose rockfill dam by far exceeds in simplicity the analysis of its inner stability. His conclusion that a compacted fill dam should be given preference where possible over a loose rockfill dam is fully justified on the basis of the experience on these two dams.

JOHN V. SPIELMAN,[5] F. ASCE.--San Gabriel Dam No. 2 (now Cogswell Dam), in its brief history, caused considerable concern to those responsibly connected with the project. Mr. Baumann's paper is of value because it describes the features of design, construction, performance, and repairs of a dam in which valuable lessons may be learned from the mistakes in design and construction, as well as in the corrections that were made.

The writer, as a design engineer with the Los Angeles County Flood Control District from 1932 to 1942, feels qualified to comment on some of the features of design and the performance record.

San Gabriel Dam No. 2 was unusual in many respects. First, in design and specifications for construction, the current practices of the time were not followed. Mr. M. M. O'Shaughnessy once described to the writer the two principal methods by which successful rockfill dams had been built. The method he preferred and used at Morena Dam[6] is to put only large pieces of clean sound rock into the fill. Sluicing is not required in this method because large stones bear directly on large stones and little settlement is likely (except perhaps in very high dams). The second method is to put most or all of the quarried material into the fill, meanwile sluicing it so thoroughly that the settlement occurs immediately during the construction period.

At San Gabriel Dam No. 2, an unfortunate composite of the two methods was followed. Sluicing was not provided for in the specifications. An attempt was made to keep out most of the quarry fines, but it was found that the grinding action of the equipment, particularly the cat treads, on the surface of each of the 25-foot high lifts, resulted in 8" to 12" of ground-up rock. If this had been sluiced down into the voids before beginning the next lift, it is believed that the principal cause for subsequent settlement would have been eliminated; but the surface was only scarified before beginning the next lift. Consequently, settlement came when the fill became wet from rainfall.

The author states the amount of settlement as a percentage of the height of fill. It may seem more impressive to express it in actual feet of accumulated settlement. In the storm of January 1, 1934, the crest at station 4 + 80 settled 5.8 feet in one night and for the six-month period from the time of first completion of the crest, about December 1, 1933, to June 13, 1934, the aggregate settlement at this point was 13.6 feet. Subsequently, the applied sluicing caused an additional drop of 3.9 feet, making a total of 17.5 feet by August 8, 1935.

The freeboard was maintained during this period by adding rock to the crest from time to time.

The original design freeboard was 20 feet. The 1938 storm filled the reservoir and produced a head of 11.4 feet on the spillway. Therefore, it might be concluded that if the 1938 storm had occurred in January of 1934, the probable settlement combined with the high reservoir level would have resulted in a disastrous overtopping of the dam.

The unusual and supposedly "more flexible" design of the laminated concrete facing with the "floater slab" connection to the cutoff never had a "fair chance" because the conditions for which it was designed were so greatly exceeded. Perhaps with a nominal amount of settlement as is experienced in most rockfill dams, this facing would have been satisfactory. It is the writer's conclusion, however, that there is no justification for more than a simple heavy reinforced-concrete slab.

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[5] Cons. Engr., San Marino, Calif.

[6] "Construction of the Morena Rockfill Dam, San Diego County, California," by M. M. O'Shaughnessy, Transactions, ASCE, Vol. 75, 1912.

The temporary timber facing applied to San Gabriel Dam No. 2 was quite successful. There was sufficient seepage so as to aid the settlement during the period of water loading and the life of 12 years was long enough for settlement to practically cease.

Finally, in 1947, a permanent reinforced concrete facing of simple, rugged design was placed on the dam.

Many existing rockfill dams have had sufficient settlement to result in serious cracking of the facing so that water loss is of consequence. Thus, San Gabriel No. 2 Dam is now, perhaps, a tighter and better dam than many that did not have the exaggerated troubles that this dam experienced.

The writer notes that San Gabriel Dam No. 1 is now labeled a rockfill dam, but on checking, finds that, despite the large central portion of compacted earthfill, over 50 per cent of the composite volume is loose rock fill; thus it is a rockfill dam according to definition. However, if the outer slopes, which are 1 on 3 both upstream and downstream, were as steep as some more recently designed dams that is, about 1 on 2.25, then the compacted earth fill would comprise over 50 per cent of the total, and it would then be considered an earth dam.

The small amount of settlement, 0.4 foot, emphasizes the vast difference between this dam and the usual rockfill dam.

F. L. LAWTON,[7] F. ASCE.--The author's paper on the 280 feet high Cogswell and the 380 feet San Gabriel dams is an unusually valuable addition to available information on rockfill dams.

The author, in referring to the cutoff wall between elevs. 2235 and 2320 of the Cogswell right abutment, noted that the placement of rockfill in the V-shaped gap from a greater height than the specified maximum of 25 feet made arching between the fill slope and the abutment wall unavoidable. Is there positive proof of this, in view of the later observation on the settlement in the previously-dry rockfill occasioned by the storm of December 31 - January 1, 1934?

The author's statement that the initial settlement of some 4 percent at the right abutment, due to the storm, was only increased to 6 percent by several months of pumping clear water, through the facing, into the rockfill is most important, particularly when contrasted with the 6 percent average settlement secured at San Gabriel dam by the use of about twice as much volume of sluicing water as rock. It can, it would appear, be concluded that ample sluicing water is imperative, a condition generally confirmed by practice on most rockfill dams.

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[7] Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

The shrinkage of rockfill after placing in Cogswell dam, during construction, as shown by Fig. 11, at approximately 18 percent must be attributed to consolidation of dry rockfill placed in lifts not exceeding 25 feet. Does the author consider the shrinkage during construction is materially changed by dry deposition in higher lifts, say 50 to 75 feet?

The deduction by the author that "--rockfill dams of major height can by no means be considered immune to pore pressure, especially in the interior of the lower part of the dam where densities would normally be greatest" is significant because of the probable formation of "a core of low permeability" in the lower portion. This appears to be confirmed by the mechanically compacted material in zone 3 of the San Gabriel dam, resulting in a percolation rate of 3 feet per year under a gradient of 1 to 1. It is also confirmed, it would appear, qualitatively by the 3 inch core from the San Gabriel dam.

The observation that the San Gabriel dam "Under the maximum water load deflected downstream 0.11 feet at the crest and returned (elastically) within a fraction of an inch to its position before the flood when the reservoir was emptied," is a confirmation of a little known property of a rockfill dam.

The experience with the upstream facing on the Gogswell dam is one more case of the difficulty of securing trouble-free performance with any rigid or semi-rigid facing.

In view of the different design of the two dams operated by the one owner, it would add to the value of the author's very fine paper if the maintenance costs for the Cogswell and San Gabriel dams over the same period of years could be stated.

CLAUDE A. FETZER,[8] M. ASCE and ELTON G. KNIGHT. [9]--The factual data presented on the performance of Cogswell and San Gabriel Dams are a major contribution in the advancement of the design of rockfill and earthfill dams. These data are particularly important because both dams have been subjected to spillway floods. The settlement and movement of the upstream face of Cogswell Dam (San Gabriel Dam No. 2) during the floods and the type and quality of rock obtained from the quarry for San Gabriel Dam No. 1 should be indelibly impressed on the minds of all dam designers.

With reference to the settlement and permeability of rockfill dams, it is conceivable that rockfill dams are not a mass of large rock with larger interstices; however, it is doubted that the voids at the base of rockfill dams are filled with high density material. The conclusions that the filler material has a density was based upon a computed density of 132 lb per cu ft for the material in the voids of San Gabriel Sam No. 2. The computed density was based upon the following assumptions:

1. Average volume per truck amounted to 11.5 cubic yards.
2. Loose rock on trucks had 40 percent voids.
3. Quarry dust equalled the amount permitted by the specifications - 3 percent of rock weight.

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[8] Chf., Design Sect., Foundation Matls. Branch, Corps of Engrs., Los Angeles, Calif.

[9] Chf., Investigation Section, U. S. Army Engr. Dist., Los Angeles, Calif.

4. Seventy-five percent of rock remained uncrushed and 25 percent of the rock combined with quarry dust to fill the rock voids.
5. Specific consolidation at the base of the dam was equal to twice the average consolidation.

No attempt will be made to evaluate these assumptions; however, the computed density of the filler material is questioned. A density of 132 lb per cu ft is a very high density, and it is much higher than the densities obtained in test fills built for San Gabriel Dam No. 1(3). In these test fills, quarry material passing a 6-inch by 9-inch "grizzly" opening was placed in 25-foot lifts and throughly sluiced at the time of placing. Densities were obtained in the fill by sinking shafts and testing the materials at 5-foot intervals. The average dry density of the fines was found to be 89 lb per cu ft, and the dry density of the rock and fines averaged 122 lb per cu ft. In comparison the densities obtained on the same material that had been placed in thin layers and rolled with a heavy roller averaged 122.2 lb per cu ft for fines and 144 lb per cu ft for rock and fines.

In the test fills built for San Gabriel Dam No. 1 the entire weight of the fill was carried by the mixture of rock and fines; whereas, in a rockfill dam it is presumed that the weight of the fill is carried to a considerable extent by point to point contact of the uncrushed rock. If this is true, the compaction of the filler material in a rockfill dam would probably be less than that obtained in a sluiced fill of fines and small rock, and would undoubtedly be less than that obtained in a rolled fill.

In conclusion it would appear that more definite information on the density of filler materials at the base of rockfill dams is needed prior to assuming that the voids are filled with high density material.

HARRIS C. PORTER,[10] M. ASCE.—The author has made a thorough summary of the features of special interest concerning the design, construction, and performance of the two rockfill dams treated in this paper. Very little remains to be added in the opinion of the writer who is in general agreement with the author's conclusions and wishes however to offer supplemental information concerning details of construction of Cogswell Dam which may be of interest to those who encounter situations of a similar nature. For that purpose this discussion is limited to rockfill, abutment preparation and cutoff and floater slab construction.

The quarry developed in Devils Canyon provided rock of excellent quality in adequate quantity to maintain the construction schedule of completion of dam fill and facing prior to the winter of 1933-34. The grading of rock to meet specification was assured by careful sorting in the quarry. Normal dumping segregation was experienced in the fill. Careful removal by power equipment of the surface accumulation of quarry chips, dust, and temporary construction road topping prior to the construction of the next lift appeared to be adequate. However, the sluicing effect of rain storms very clearly showed otherwise. The immediate settlement observed when December storms wet down the dry dumped fill emphasized most drastically the importance of sluicing rock fill during construction. Also it has been noted that settlement varied greatly in adjacent portions of the fill. It is the belief of the writer the variation in settlement was due in part to the irregular slope of the south abutment as may be seen in Fig. 2 and Fig. 3(b) and in part to construction methods used to avoid delay of the rock fill. An analysis of the profile on center line of crest and of sections across the south abutment ridge indicate the fill was subjected to the effect of a considerable variation in slope of abutment surface. In addition the depth of fill varied considerably both in longitudinal and transverse section. These factors contributed materially to the uneven rates of settlement.

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[10] Assoc. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

During early stages of construction the abutments were sluiced to remove loose over-burden prior to the start of rockfill. Abutment excavation and trimming was continued on each abutment above the fill and was generally well in advance of the rockfill except in a section of the right or south abutment above elevation 2230 referred to by the author. Here the amount of abutment excavation necessary to comply with requirements was greater than had been estimated. As a result, the rockfill, which progressed at a rate of approximately 75,000 cubic yards per month, got ahead of the right abutment cleanup. By means of stage construction, with increments of more than one 25 ft. lift underway at one time, delay was avoided and fill lifts were nearing completion at the south abutment contact as successive sections of the abutment stripping met the requirement. In this manner construction delay was avoided and specifications as to height of lift were observed. Even so the piecemeal construction of the fill adjacent to the south abutment, (detail of which is too involved for review in this discussion), resulted in conditions within the fill and packed rock which could only further complicate the settlement problem.

Approval of cutoff alignment and excavation was withheld pending completion and approval of abutment stripping. To illustrate the situation, when the fill and packed rock were above elevation 2375 the cutoff concrete in each abutment was at elevation 2240. Excavation of the cutoff in the reach involved on the south abutment, elevation 2230 to 2350, was on the face of a steep cliff. The bench and trench excavation required was such that the cliff above the cutoff wall was in an unstable position tending to be under-cut by the open trench excavation. As a means to reduce the degree of hazard and to keep pace with the construction of fill and face slabs and avoid possibility of an extensive delay in the completion of the dam, the cutoff trench was filled with concrete with provision for future deepening as necessary to satisfy the requirement of the State Engineer. Also at this time drilling and grouting of cutoff included the unfinished section. Fourteen holes were drilled to depths of 150 feet within the cutoff limits in question and were grouted with a total of 154 sacks of cement.

During 1935 the unfinished section of cutoff wall was completed. The excavation beneath concrete was made by inclined stope and shaft. An adit along the side of the concrete cutoff provided access beneath the previously placed concrete. The inclined tunnel followed beneath the concrete for a distance of approximately 160 feet on slope. Access to the upper end was accomplished by raising to the surface through a shaft some 35 feet along the outer face of the cutoff previously concreted. The work involved about 320 cubic yards of tunnel excavation to develop access to the unapproved cutoff and to intercept seams in question and remove unsatisfactory rock. The incline and shaft were filled with concrete. Approval of the cutoff in June of 1935 was followed by additional drilling and grouting. A total of 33 holes were drilled and grouted consisting of 27 holes of 7" diameter varying in depth from 100 feet to 80 feet and 7 holes of 4-1/2" diameter and depth 80 feet. The average depth of drilling in rock below concrete was approximately 40 feet. Each hole was grouted in a continuous operation working from the bottom up through a 1" pipe insert with pressure starting at 25 psi and increased to 100 psi at refusal. 785 sacks of cement were used in the grout placed at that time which concluded the cutoff construction.

A change of plan during construction affected the joint between face slab and cutoff. In Figure 4(d) a typical detail of cutoff wall floater slab and face slab connection is illustrated. The writer invites attention to the provision for "small round stone" to be placed under the hinged floater slab. The flexibility intended in this design would quite likely have been greater than actually experienced in the hinged slab as built. The gravel fill material was omitted and a portion of the floater slab was placed over fill consisting of fine quarry run of sizes ranging from chips to 1 cubic foot. Other portions of the floater slab were supported by packed rockfill. This may account in part for failure of the hinged slab to follow the movement in the fill. However, the extent to which the change in fill material beneath the floater slab may have affected the flexibility of the hinged joint would be difficult to evaluate. Also because of the amount of settlement experienced it is considered doubtful that other conditions of fill under the floater slab would have materially changed the extent of damage to the connection between face slabs and cutoff wall.

In conclusion, in the opinion of the writer, the beneficial effect of sluicing rockfill cannot be overstressed. It may also be noted that such an opinion is in essence a restatement of comments in discussion by other writers. (References are those given by the author.) In retrospect it seems evident now that a number of factors contributed to the failure of face slab experienced at Cogswell Dam. The dry fill construction method, admittedly not the only cause of excessive settlement of fill, certainly was the greatest single factor. Even a fill of clean large rock when properly sluiced is given a degree of beneficial lubrication and settlement during construction is aided. Where fines are present sluicing is indispensable. It is believed to have been clearly shown that adequate sluicing of the fill during construction of the dam would have settled the fill at the time of construction to a condition which would have made the construction of a permanent concrete face possible. And as an additional illustration, later experienced at San Gabriel Dam where the dumped fill was sluiced with two parts of water to one rock by volume further demonstrated the beneficial value of sluicing by the amount of initial settlement observed during construction.

Discussion of the cutoff construction problem experienced and its solution is offered in the hope that should a situation of a similar nature occur on other construction it may be of benefit to those involved to know of previous experience. The method used was well justified both through elimination of a dangerous excavation hazard and by the avoidance of construction delay. The adequacy of the cutoff and grouting has been well demonstrated during the years since construction. Other than a blanket of gunite the abutment has required no attention since completion of the dam.

PAUL BAUMANN,[11] F. ASCE.—The writer would be very happy indeed if he had a direct and unequivocal answer to all the pertinent and constructive questions raised by the discussors. Unfortunately, however, questions raised in regard to the actual physical performance of the rock in the interior of the dam must of necessity be answered indirectly, that is to say, in light of circumstantial evidence. One feature, however, should be made clear to avoid further misunderstanding, namely that Cogswell Dam itself is not considered to be of critical height but that the lesson learned through its performance should be duly considered by engineers engaged in the planning and design of rockfill dams of materially greater height. This happens to be the trend at the present time. There is a great challenge in the harnessing of formidable energies through the construction of major dams but the responsibility is, needless to say, even greater. The writer's paper, so ably and gratifyingly discussed, was prepared in the spirit of this responsibility. It does sound a warning in regard to the possibility of transformation of a loose rockfill into a dense rockfill with increasing height. In the writer's opinion, this possibility had not been duly considered heretofore.

Before referring to each of the discussions separately a general statement concerning pertinent factors in the construction of Cogswell Dam may be in order. It is hoped that some of the questions, implied rather than expressed by discussors, may be answered at least in part.

The writer was aware of and intrigued by the performance of Cogswell (then San Gabriel No. 2) Dam during its construction before his employment in October 1934 with the Los Angeles County Flood Control District. He strongly felt that the causes of measured shrinkage during construction and additional large settlement when the completed embankment was wetted by some 15 inches of rainfall, merited thorough investigation, study and analysis. Thus, immediately after his employment with the District in charge of dams and water conservation activities, the writer assigned his principal assistant, the late N. B. Hodgkinson, A. M. ASCE, formerly in charge of the District's design division, to undertake such investigation and report thereon. After examination of several hundred inspectors' reports, covering all phases of construction activities from the quarrying of the rock to its final placing in the dam, it was concluded that 1) the size and grading of the rock all the way down to quarry dust closely conformed to specifications; the average volume per truck load of Class A rock placed throughout the main fill of the dam was 11.5 cu. yds. and the corresponding weight 33,600 lbs.; the average weight per truck load of quarry dust was estimated at 1000 lbs. leaving a net weight of 32,600 lbs exclusive of quarry dust. These are the figures used by the writer in arriving at densities after consolidation. Frankly, the porosity of 40% exclusive of quarry dust which follows from the above figures and the specific gravity of 2.80 as an average of 211 tests, appeared rather low at first glance. However, even if the porosity had been in error by possibly 10% and therefore had been

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44% instead of 40%, the conclusion in regard to impermeability at the base of the dam would remain substantially unchanged. The dry density of the crushed material (which of course includes rock dust) and quarry dust would be reduced to 123 p.c.f. or slightly more than the 120 p.c.f. required in the center core section of San Gabriel Dam (No. 1). The rate of percolation for a unit gradient corresponding to this density was found to be 3.0 ft. per year. Hence, the question of whether or not a rockfill dam will become impervious from the base upward with increasing height does not hinge so much on initial densities as it does on the size and uniformity of the rock. Mr. O'Shaughnessy, as quoted by Mr. Spielman, has put the wisdom of the fundamental requirement for dry construction of rockfill dams in a nut shell, namely that the rock should be uniformly large. While crushing of points in contact would still result in shrinkage, it would be small as compared to that of rock varying in size such as specified for Class A at Cogswell Dam. The reason is simple. In addition to point crushing, a mixture of large and small rocks gives rise to what might be called the "nut cracker effect". Small rocks wedged between large rocks can readily be crushed. It very seldom works the other way. Therefore, to minimize such crushing and production of fines associated therewith, the small rocks together with initial fines should be sluiced into the interstices at time of placing. In other words, sluicing of such mixtures is indispensible if excessive production of fines after placing is to be avoided.

Unfortunately, the ideal rockfill dam, consisting of uniformly large rock, is seldom economical because of the smaller material unavoidably produced in blasting that will have to be separated and wasted. The writer knows of no major rockfill dam built exclusively of uniformly large rocks. The cost of sluicing a mixture of rock with a minimum of waste evidently has so far been considered economically advantageous. Nevertheless, the possibility of creating a zone of low permeability in the interior even of a sluiced dam in excess of say 350 feet in height should always be carefully investigated. "Low permeability" should be interpreted as not free draining. Hence, even if some channels here and there should remain open the respective percolation could be far more critical than that through a free draining mass.

Mr. Brown deplores the absence of physical demonstration of impermeability within the body of a high rockfill dam and so does the writer. If San Gabriel Dam (No. 1) had been built in a similar manner as Cogswell Dam, chances are that such demonstration would have been convincingly furnished, even if the rock there had been of equal quality as that at Cogswell. Actually not all of the rockfill dams built in the past have been favored with the same high quality of rock as that at Cogswell Dam, nor will future dams necessarily be so favored. The original estimated leakage of 130 c.f.s. through Cogswell Dam under full head has been revised to 105 c.f.s. Based on free draining conditions and the timber facing the controlling "water stop", 200 c.f.s. were anticipated. There were no means of locating the free water surface for turbulent flow or the phreatic surface for laminar flow. The void space between the extra large (Class B) rock at the downstream face, especially in its lower part, prevented any observation of the discharge face of flow or percolation through the interior zone of the dam. Furthermore, the tightest part of this zone would, in view of normal pressure distribution in vertical sections, be below streambed. However, a few corresponding water surface elevation and leakage points taken from the official record, may be of assistance in arriving at some conclusion as to the effect on leakage of consolidation of the rock mixture.

Water Surface Elevation	Date	Leakage (c.f.s.)	
2177	12-10-37	1.00	
2236	2-1-38	6.00	
2337	2-26-38	20.00)	Over 20" of rainfall on exposed part of timber facing
2350	3-1-38	40.00)	
2397	3-2-38	105.00)	
2312	3-10-38	6.50)	Timber facing "sealed" by sediments
2357	3-20-38	13.00)	
2376	6-30-38	15.00)	
2170	8-16-38	0.50)	

For a free draining rockfill, the leakage through the dam should be in direct proportion to the pressure on the facing the area of which can be approximated by a trapezoid 410 feet wide at the top, 185 feet at the bottom and 260 feet in height. The leakage Q' per unit width at the maximum height of the dam

$$Q' = c \; \frac{62.5H^2}{2}$$

in which c = coefficient of leakage (affected by the swelling of the timber facing)

Since the pressure distribution is triangular it follows that 1/2 Q' must pass through the bottom 1/3 of the section. Taking the trapezoidal shape of the facing into account it follows that the total leakage for maximum head

$$Q_{max} = 105 = 7800 \; c \; H^2 \text{ and}$$

$$c = 0.019 \times 10^{-6} \text{ ft per sec}$$

From the above tabulation it may be noted that for the water surface at elevation 2337, that is at 3/4 H the leakage was 20 c.f.s. whereas under this head the coefficient

$$c = \frac{20}{4600 \; H^2} = 0.00614 \times 10^{-6} \text{ ft per sec}$$

Thus it follows that about 75% of the maximum leakage passed through the top 1/4 of the facing.

If, conversely, percolation through the consolidated rockfill had been controlling the leakage and the rockfill had become a quasi homogenious fragmental mass, similar to Zone 3 at San Gabriel Dam then, as shown in the flow net[12] 1/2 of Q" would have passed through the top 1/3 of the maximum section of unit width. Approximating the flow through the top, middle and bottom 1/3 of the section by 1/2, 1/3 and 1/6 of Q" respectively then

$$\frac{q''_t + q''_b}{2} \times H = Q''$$

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[12] "Design and Construction of San Gabriel Dam No. 1," by Paul Baumann, M. ASCE, Transactions, ASCE, Vol. 107, 1942, p. 1631.

in which q_t'' = rate of flow at top and q_b'' = rate of flow at bottom.

Applying the trapezoidal shape of the face as before the rate of flow at the top, middle and bottom

$$q_t = 717.5\ Q''/H$$

$$q_m = 297.5\ Q''/H$$

and

$$q_b = 46.3\ Q''/H \text{ (due to averaging flow through bottom 1/3)}$$

There follows[13]

$$Q_{max} = 313\ Q''$$

and for

$$Q'' = \frac{5}{16}\ KH$$

$$Q_{max} = 105 = 98\ KH$$

Therefore, for H = 260'

$$K = 0.004125 \text{ ft/sec} = 130{,}000 \text{ ft/yr}$$

For the water surface at 3/4 H and Q = 20 c.f.s.

$$Q = 20 = 157.5\ Q'' = 48.2\ KH$$

and

$$K = 0.001595 \text{ ft/sec} = 38{,}700 \text{ ft/yr}$$

Hence, in this case the permeability for the bottom 3/4 of the dam would be about one-fourth of that for the entire height of the section.

While no definite conclusions could be drawn from the above one thing appears certain, namely that the lower part of the dam was less pervious than the upper part.

Mr. Brown's comparison of the massive core section at San Gabriel Dam and a hypothetical one 10 to 20 feet thick with correspondingly steep faces merits close attention by the profession. The inner flexibility and unpredictable attendant movements inherent with all rockfill dams call for core dimensions commensurate with differential (unbalanced) pressures which must be taken up in shear by the saturated core material. Again, this problem grows rapidly with the height of the dam. The effect of weighting on the roughly 1 to 1 slopes of the core section of San Gabriel Dam (No. 1) was treated as prestress by the writer and its significance was analyzed.[14]

Mr. Spielman ponders the qualification of San Gabriel Dam as a rockfill dam. Although he took an active part in the revision of the original design he apparently forgot that the "large central portion" is compacted quarry rock with pieces up to 9 in. in size rather than compacted earthfill. The earthfill part in Zone II amounts to only about 10% of the total volume. Hence, "rockfill dam" seems to be the proper classification. Incidentally, Mr. Spielman also deserves considerable credit for the ingenious design of the temporary timber facing that performed to full satisfaction at Cogswell Dam.

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[13] Ibid., p. 1630, Eq. (11b).

[14] "The Significance of Prestress in Embankment Dams," Proceedings, 2nd Conference on Soil Mechanics and Foundation Engineering, Vol. VII, p. 92.

The writer agrees with Mr. Spielman and so stated, namely that the laminated facing and flexible connection to the cut-off wall by means of the floater slab might well have performed satisfactorily under normal settlement conditions. However, the fulcrum effect along the abutments could still have been critical even if settlements had been considerably smaller. To overcome this effect would have required much wider benching between the cut-off wall and the cliff face so as to rest the entire floater slab on a tapering section of "small round stone" with its thickness roughly proportional to the distance from the hinge. (See Fig. 4d) This would have required greatly increased rock excavation. Mr. Spielman's conclusion that "there is no justification for more than a simple, heavy reinforced-concrete slab" is therefore strongly concurred in.

Mr. Lawton's question regarding arching of the rock in the V-shaped gap adjacent to the right abutment must again be answered based on circumstantial evidence as follows: (1) The higher lift naturally caused greater segregation of the rock, the larger pieces running ahead of the smaller ones. At the bottom of the gap the rocks hit a pocket which actually was semi cone-shaped, in that only the upstream portion of the dam was affected by the delay in the cut-off wall construction while progress on the portion downstream therefrom continued as before. Thus, with large rock falling into this semi conic pocket arching could hardly have been avoided. (2) The maximum settlement due to rain and subsequent sluicing occurred in this V-gap area and is in plain evidence today. In fact Fig. 12 shows it as a sag in the cat walk at the top of the paneled facing slab.

Mr. Lawton's statement that "ample sluicing water is imperative" cannot be too strongly emphasized, especially where rock mixtures are concerned such as described under Class A rock. The main part of rockfills are predominantly of this kind of mixture.

As to the wisdom of lifts up to 50 to 75 feet instead of the 25 feet used at both Cogswell and San Gabriel Dams the writer can see no decisive advantage in higher lifts, whether placed dry or wet, but can readily conceive of some disadvantages. These are due to (1) segregation of larger from smaller rocks previously mentioned; (2) lesser benefit from equipment compaction and (3) increased break-down and production of fines due to greater impact force.

The elastic behaviour of San Gabriel Dam (No. 1) under full water load and subsequent emptying of the reservoir can only be attributed to the monolythic properties acquired by the massive, center core section of screened and compacted quarry rock. In addition to the initially created high density this section's roughly 1 to 1 slopes are weighted and laterally supported by loose rockfill and therefore are prestressed as outlined above. Under these conditions deformations can become elastic. This further strengthens Mr. Brown's statement regarding thin cores in rockfill dams. In brief a massive core may well be the strongest and a thin core the weakest part of a rockfill dam.

As to comparative cost of maintenance, repairs and replacement at the two dams the records show that during the last ten years some $25,000.00 average were spent at each dam each year. Since the initial costs were, in round figures, $4.5 and 17 million for Cogswell and San Gabriel Dam, respectively, the corresponding yearly cost of maintenance, repairs and replacements amounted to 0.56% and 0.15% of the initial investment.

Messrs. Fetzer and Knight enumerate five "assumptions" upon which the computation of the density of 132 p.c.f. was based. In light of the introductory statements in this closure the first three items quoted can not be classed as assumptions, whereas the fifth item is in that category only so far as possible axial arching at the base is concerned. In the absence of such arching the pressure at the base of a column of rock is twice the average and so would normally consolidation be. In a symmetrical monolyth of triangular cross section the major and minor principal stresses follow circular arcs and radial lines, respectively, around and through the apex of the triangle so long as the loading is due to gravity only. Normal stresses are uniform along horizontal planes. Since shear is zero along principal stresses there is no shear along the axis of symmetry. In comparison the pressure distribution along horizontal planes of a symmetrical triangular embankment dam of compacted earth is directly proportional to the height of the column above it. Hence, basically it is triangular. However, due to plastic deformations the stress distribution has never been uniquely established but has merely been approximated by applying a "compaction factor" which is the ratio of horizontal and vertical stress. Therefore all stresses at points on the outer slopes are zero and the maximum concentration of stress or pressure occurs in a zone at and adjacent to the axis of symmetry. In general this no doubt applies to rockfill dams as well. The writer referred to this zone as the interior portion of the dam.

The writer agrees that there is room for debate on item four which is an assumption entirely based on judgment and not on observation. In the well graded mixture of Class A rock, 40% varied in size from quarry chips to 1000 pound rocks. Thus the 25% of the total assumed to have been crushed was generally smaller than 1000 pound rocks, more of the order of 500 to 600 pounds maximum, that is rock of about three cubic feet in volume. Considering the fact that 60% of the total volume was rock between 1000 and 14,000 pounds the "nut cracking" of 25% of generally much smaller rock does not seem to be an unreasonable expectation.

The test fills at San Gabriel Dam (No. 1) should not be used as a criterion of density at the base of Cogswell Dam. The dry density of the fines of 89 p.c.f. quoted by Messrs. Fetzer and Knight were found in the test fill, 25 feet in total height, between its surface and twenty feet below the surface. Hence, the self-compacting effect under some 280 feet of fill was not reflected in this test fill. The shrinkage in the dam for the first 25 feet lift was 1.75%. Thus the composite dry density had increased from 105 to 106.8 p.c.f. which would have no effect on permeability.

The uncertainty of stress distribution in an embankment dam of compacted earthfill is much greater for rockfill. There is simply no way of even approaching the exact distribution of internal stresses in a rockfill dam. As stated by Messrs. Fetzer and Knight the weight of the fill as well as the applied load is, to a considerable extent, probably carried to the foundation by point to point contact. However, this does not mean that only the larger rocks are in contact unless the fill consists of nothing else. In a mixed fill some of the smaller pieces will necessarily get between the larger rocks, especially if placed dry. Hence, there will be rock crushing first, point crushing second and compression of filler materials last by general order of sequence as visualized by the writer. The compressed filler material, whether sluiced or crushed into the interstices will lend stability to the larger rocks subjected to excentric contact loading. It would be interesting indeed if means could be devised for direct observation of the goings-on in the interior of a rockfill during construction and subsequent application of water load. It would eliminate the need of much speculation.

In elaborating on some of the construction features Mr. Porter has furnished much valuable supplemental information. Of special interest is the description of the completion of the cutoff wall on the right abutment. In the paragraph next to the last Mr. Porter enumerates what might be called the cardinal principles of rockfill dam construction with special emphasis on the importance of sluicing. The writer is in full agreement with these principles but would like to reiterate his word of caution in regard to the ever increasing height of rockfill dams, even if thoroughly sluiced, namely, 1) the face slopes should be decidedly flatter than the angle of repose of the rock; 2) if a water tight concrete facing is intended a temporary timber facing pending cessation of settlement may be advantageous; 3) if a watertight core is intended it should add to the stability of the dam rather than relying on the dam for its support; and 4) assurance of free drainage of the fill should not be taken for granted but should be carefully analyzed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3065

ROCKFILL DAMS: SALT SPRINGS AND LOWER BEAR RIVER CONCRETE FACE DAMS

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With Discussion by Messrs. Carlos Tercero; Geoffrey Davey; Masatoshi Kawase; Tatsuo Mizukoshi; F. L. Lawton; C. M. Roberts; Karl Terzaghi; and I. C. Steele and J. B. Cooke

SYNOPSIS

The 28-year service record of Salt Springs Dam, in terms of settlement, leakage, face cracks and maintenance is presented. The troubles at Salt Springs are traced to specific construction weaknesses.

The design, construction and performance of the two Lower Bear River Dams is presented and compared to Salt Springs Dam.

Note.—Published, essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1737. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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INTRODUCTION

Prior to 1925, there existed only eight rockfill dams higher than 100 feet; Morena, Strawberry and Swift Dams all being highest at about 160 feet. The eight dams were all in the United States and most in California. In 1925, a 115-foot step in height was made with the 275-foot high Dix River Dam of the Kentucky Utilities Company. This dam was a major step in the development of concrete face dams and Mr. L. F. Harza and Mr. G. W. Howson deserve particular credit. The next increase in height was made six years later in 1931 with a 53-foot step from the Dix Dam to the *Pacific Gas and Electric Company's 328-foot Salt Springs Dam. Though impervious face dams in the 200 to 300-foot range in height have since been built, Salt Springs Dam remained the world's highest until 1958, a period of 26 years. In 1958, a 27-foot step was made to the 355-foot high Paradela Dam in Portugal.

Since Salt Springs was completed in 1931, impervious core rockfill dams of various types have been developed and have been constructed to continually increasing maximum heights that presently are in the range of 400-450 feet. The economics and the high degree of safety, characteristic of rockfill dams of all types, have resulted in their increased use. The need for always higher dams of unquestionable safety makes careful study and reporting of the then existing high dams an important obligation of the engineers engaged in their design, construction and operation.

The P. G. and E. Co. serves Central and Northern California. The electric production comprises 14 thermal electric plants and 58 hydroelectric plants having a total capacity of 4,533,000 kw. The hydro plants are nearly all located in the Sierra Nevada Mountain Range which extends some 430 miles along the eastern boundary of California. Except in the northern end of the Sierras, where the mountains are of volcanic origin, the basic formation is granitic and there is little natural surface or underground storage. Precipitation is in the form of winter snow which melts in the late spring and early summer. Annual and, in some cases, cyclic storage is necessary to provide dependable hydroelectric power. To provide the storage, and to provide diversions and afterbays, P. G. and E. Co. has 106 dams of all types. With minor exceptions, these dams are utilized primarily for power purposes, but provide important benefits to irrigation and domestic water supply, flood control and recreation.

Fourteen of the 106 dams are of the impervious face rockfill type. The earliest, Fordyce Dam, dates back to 1873 and the most recent, Wishon and Courtright, are being completed in 1958. Experience with all of these dams has been very satisfactory and all of them are in service today. Development of the impervious face rockfill dam has been appropriate and particularly economic in the massive granite formations of the Sierras.

The early impervious face rockfill dam experience of P. G. and E. Co. may be considered to have culminated with completion of the 328-foot high Salt Springs Dam in 1931. Twenty-one years passed before the Company constructed the next concrete face rockfill dams, the Lower Bear River Dams in 1952. The 21 years of experience with Salt Springs as well as experience with other rockfill dams was effectively used in the design of the Lower Bear River Dams.

This paper will bring Salt Springs Dam experience up to date and will review the construction from the standpoint of its influence on the performance

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*Hereinafter referred to as P. G. and E. Co.

of the dam. The paper will also present the design, construction and performance of the Lower Bear River Dams and make comparisons with Salt Springs.

Early Pacific Gas and Electric Company Rockfill Dams

The nine impervious face rockfill dams of the Company that were constructed before Salt Springs have been termed the early dams. Data on all of P. G. and E. Company's impervious face rockfill dams is presented in Table 1. All are still in service after 28 to 85 years of satisfactory performance. Experience with the six of these that are higher than 75 feet will be briefly reviewed. It was the early experience with these and other dams, together with that of Dix River Dam, that led to the adoption of a concrete face rockfill type dam for Salt Springs.

Upper Bear River Dam is now 58 years old and is of particular interest because of the steep slopes with a very thin layer of placed rock. The placed rock is one layer of hand and small derrick placed stones, and the fill is a mixture of small rock and fines. The face was timber with no concrete backing. The up and downstream slopes are the same. The 0.50:1 slope of the upper 25 feet and the 0.75:1 slope of the lower 50 feet are considerably steeper than the natural dumped rock slope of about 1.3:1. The thin layer of dry placed rock is certainly not adequate as a retaining wall. The rockfill should not have as high a shearing strength as a modern fill of dumped and sluiced large rock.

This dam and other early rockfills with steeper than natural dumped slopes have provided evidence that the shearing strength is much greater than that based on a 1.3:1 natural slope. For example, a rockfill with an infinite slope of 0.5:1 must have a shear coefficient exceeding 2.0, whereas a 1.3:1 natural slope indicates a shear coefficient of 0.77. A coefficient of 1.0 is commonly used in design of core rockfill dams. The natural dumped slope of a rockfill is not an indication of its shear coefficient.

In 1929, a dumped fill at natural (1.3:1) slope was added to the downstream face and the spillway enlarged. In 1932, the crest was raised with 5 feet of placed rock to obtain increased storage. In 1953, the deteriorated 50-year old timber was removed and a continuous gunite face with no joints was applied to the placed rock. The gunite thickness varied from 3 inches at the top to 5 inches at the base of the dam and reinforcing from one to two layers of No. 4 x 4 by No. 6 x 6 mesh.

Meadow Lake Dam is now 55 years old, and its continuous service further substantiates the stability of face slopes considerably steeper than the "angle of repose". The slopes are similar to those of Lower Bear River Dam but the placed rock is even a thinner single layer. Cutoffs are only one or two feet into granite bedrock. In 1930, a forest fire burned the timber face off the dam. A gunite face with no joints, and thickness of 2 inches at top to 4 inches at the base, was installed. One vertical crack developed in the 750 feet of crest length. It was covered by a copper joint and further opening has caused no leakage. After 27 years, the face is in good condition, even though it is at elevation 7700 and exposed all winter. Leakage is negligible. The use of thin continuous gunite without joints is considered economical and successful for both dams.

Relief Dam is now 48 years old. This 140-foot high dam has not contributed much to the technology because of the very thick section of placed rock. Though the upstream face slope is all at 0.5:1, all the rock upstream from the crest is placed. This use of placed rock was probably economical in 1910, but present costs do not justify the saving of dumped rock by use of placed rock. The 250-foot high Malpaso Dam in Peru is of nearly the same section and construction as Relief Dam. The crest of Relief is curved downstream to give a dam of minimum yardage. Downstream movement opened a vertical crack near the center of the dam. This crack was covered by a copper joint and has continued to open. This is another example of the characteristic of the impervious face rockfill dam to concentrate future movement at the initial joints or cracks. This is a very favorable characteristic.

Fordyce Dam was raised in 1927. The upstream slope of the raised dam has an 18-inch slab over 1:1 loose rockfill with nominal placed surface rocks for the lower 93 feet, and a 12 to 18-inch slab on 1:1 slope over 4 to 6-foot thick placed rock for the upper 47 feet of the raise. The successful service and small settlement of this 140-foot dam, with 1:1 upstream slope and nominal placed rock gives further confidence in the high degree of stability characteristic of dumped rockfills.

Main Strawberry Dam is now 42 years old. The slopes of this dam are 1.2:1 upstream and natural (1.3:1) downstream. Placed rock is 4 to 8 feet thick, and the fill is dumped over about 20-foot depth of streambed material. The dam is at elevation 5620 and water and frost action has deteriorated some areas of the 9 to 15-inch concrete face to the extent that at some future time repair will be required. Gunite will probably be used. This dam, with its thin placed rock and natural or steeper slopes, was a very progressive design for one of the highest dams of its type in 1916. Morena and Swift Dams (Montana), 1912 and 1914, are of about the same height.

Bucks Creek Dam is now 30 years old and has given trouble-free performance. The slopes are conservatively flat, 1.4:1 upstream and 1.5:1 downstream. Placed rock is thin, 3 to 6 feet thick. The concrete slab has no joints and reinforcing steel is continuous. The face has developed no cracks, leakage is negligible and no maintenance has been required in its 30-year life to date.

Summary

These six early impervious face rockfill dams, of 75 to 140-foot heights and ages of 30 to 85 years, have given excellent service with nominal, if any, maintenance except for replacing timber face by gunite in two cases. There has never been any question as to safety. They have provided experience that has led to the confident construction of higher dams.

Engineering Significance of Salt Springs Dam

Salt Springs Dam was completed in 1931. It was the world's highest concrete face rockfill dam, 328 feet high, until the completion in 1958 of the 355-foot high Paradela Dam in Portugal. The performance of the dam has been reported in the engineering literature.(21,22)[3] However, in reviewing the literature in the light of later experience with the Lower Bear River and other dams, it is considered that the existing literature could be misleading. It could be inferred from the literature on Salt Springs that face cracking, rather high settlement movements, and maintenance are a problem and that they might be expected of such a dam. Actually, these troubles with Salt Springs have never in any way given concern as to safety, cost or continuous operation. The cost of maintenance has been negligible compared to the economy in the use of the concrete face rockfill dam at the Salt Springs site.

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[3] Numerals in parentheses, thus (1), refer to corresponding items in the Bibliography.

Study of the troubles at Salt Springs has led to the design of concrete face rockfill dams of both greatly improved service record and lower cost. If Salt Springs Dam were to be constructed with today's knowledge, it would be of more simple and lower cost design and, at the same time, give superior service.

Lower Bear River Dams were designed and constructed with the help of 20 years more rockfill dam experience than was available for Salt Springs and particularly with the help of the very detailed knowledge of the performance of Salt Springs Dam. The design of the Lower Bear River Dams included features intended to avoid the face troubles that occurred at Salt Springs and these were successful. However, the Lower Bear River Dam performance, with nominal cracks and leakage and small settlement, was considerably better than expected. This fact led to a careful review of Salt Springs construction. It was evident that the superior construction of the dumped fill at Bear River was the main reason for the improved performance. Comparative physical data is presented in Table 2.

In advance of presenting and comparing Salt Springs and Lower Bear River Dam data, general conclusions are that:

(1) The Salt Springs face cracks can be traced to specific construction weaknesses in the dumped fill.
(2) The most significant difference in construction was in the sluicing.
(3) In addition to construction changes, minor design changes have and can tend to eliminate the Salt Springs face troubles.

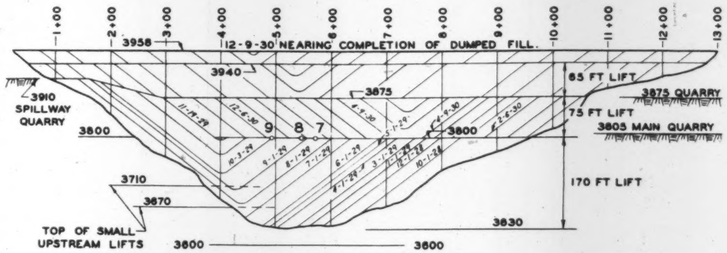
Salt Springs Dam continues to give unquestionably safe and economic service. However, to the dam engineer it has made its contribution as a laboratory for pioneering high rockfill dams. Future concrete face rockfill dams should be patterned after the design and experience of more recent rockfill dams.

Construction of Salt Springs Dam

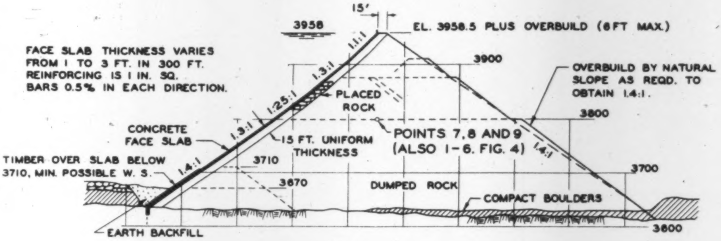
Construction began in 1928 and the dam was completed in time to store the spring runoff of 1931. The section and profile of the dam are shown on Fig. 1 and the dumping of rock is shown on the photo, Fig. 2. In 1946-47, spillway radial gates and a coping wall were installed to obtain additional storage. The photo of Fig. 3 shows the completed dam. The construction of the dam is well described and illustrated in references (1) through (20). This paper will review construction briefly and primarily from the standpoint of its influence on the performance of the dam.

Foundation and Cutoff

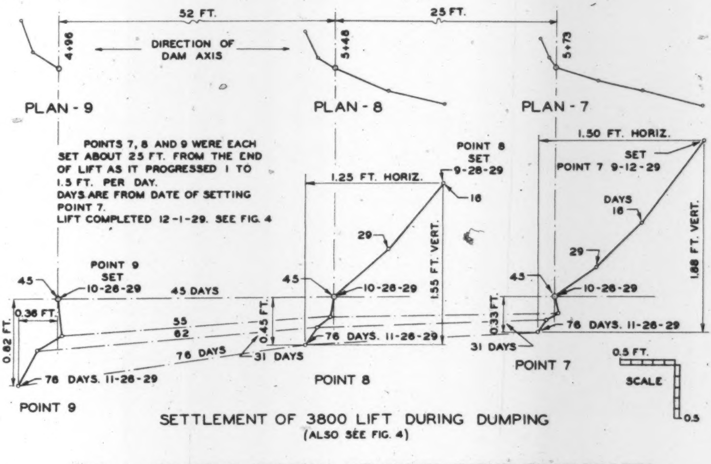
Streambed and Abutment.—A total of 312,000 cu. yds. of streambed sands and gravels were removed from the floor of the narrow canyon. All material, 28 to 40 feet deep, was removed upstream from the axis. Downstream from the axis 20 to 28 feet of fine material was removed and 10 to 15 feet


1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
11+00
12+00
13+00
3958
12-9-30 NEARING COMPLETION OF DUMPED FILL.
3940
3875
65 FT LIFT
3875 QUARRY
3910
SPILLWAY
QUARRY
75 FT LIFT
3805 MAIN QUARRY
3800
9
8
7
3800
11-19-29
12-6-30
4-9-30
5-1-29
4-9-30
2-6-30
10-3-29
9-1-29
8-1-29
7-1-29
6-1-29
4-1-29
3-1-29
11-1-29
12-1-28
10-1-28
3710
170 FT LIFT
3670
3830
TOP OF SMALL
UPSTREAM LIFTS
3600
3600

PROFILE ON AXIS (LOOKING DOWNSTREAM) - LIFTS AND CONSTRUCTION SEQUENCE


15'
3958
EL. 3958.5 PLUS OVERBUILD (6 FT MAX.)
FACE SLAB THICKNESS VARIES
FROM 1 TO 3 FT. IN 300 FT.
REINFORCING IS 1 IN. SQ.
BARS 0.5% IN EACH DIRECTION.
1.1:1
1.3:1
1.25:1
1.3:1
1.4:1
3900
OVERBUILD BY NATURAL
SLOPE AS REQD. TO
OBTAIN 1.4:1.
PLACED
ROCK
3800
CONCRETE
FACE SLAB
15 FT. UNIFORM
THICKNESS
POINTS 7, 8 AND 9
(ALSO 1-6. FIG. 4)
1.4:1
TIMBER OVER SLAB BELOW
3710, MIN. POSSIBLE W. S.
3710
3700
3670
DUMPED ROCK
COMPACT BOULDERS
EARTH BACKFILL
3600

MAXIMUM SECTION



SETTLEMENT OF 3800 LIFT DURING DUMPING
(ALSO SEE FIG. 4)

FIG. 1.—PROFILE, SECTION AND SETTLEMENT OF MAIN LIFT DURING CONSTRUCTION - SALT SPRINGS DAM

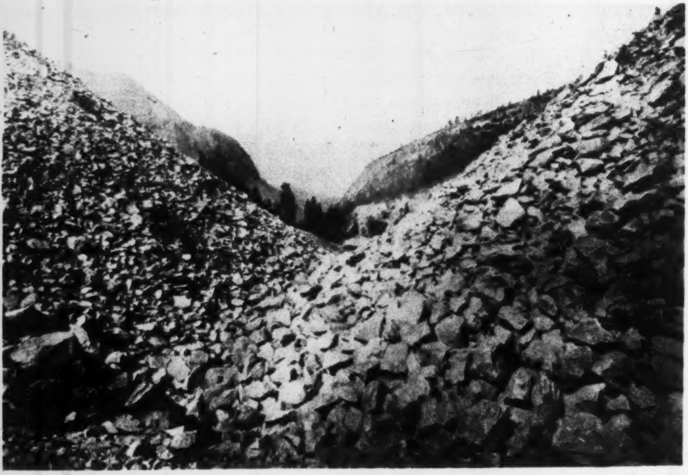


FIG. 2.—THE MEETING OF 3910 LEFT ABUTMENT AND 3800 RIGHT ABUTMENT LIFTS—SALT SPRINGS DAM

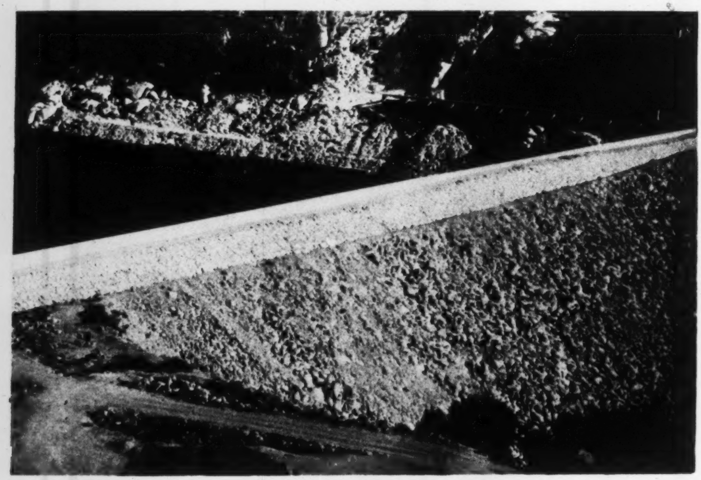


FIG. 3.—COMPLETED DAM WITH SPILLWAY GATES AND COPING WALL ADDED IN 1947—SALT SPRINGS DAM

TABLE 1.—IMPERVIOUS FACE ROCKFILL DAMS OF THE PACIFIC GAS AND ELECTRIC COMPANY

NAME	YEAR COMPLETED	* HEIGHT	FACE SLOPES HORIZ. TO 1 VERT.		PLACED ROCK THICKNESS FT.		TYPE OF FACE
			UP STREAM	DOWN STREAM	TOP	BOTTOM	
FORDYCE (INITIAL)	1873	93	.25	.25	LAYER	LAYER	MASONRY
" (ENLARGED)	1927	140	1.0	1.35	4	6	CONCRETE
LAKE STIRLING	1877	25	1.0	0.5			
LOWER THREE LAKES	1896	30	1.25	1.5	2	3	CONCRETE
UPPER BEAR RIVER	1900	75	0.5, 0.75	0.5, 0.75	LAYER	LAYER	WOOD
" " "	1932	80		1.3			WOOD
" " "	1953	80		1.3			GUNITE
MEADOW LAKE	1903	74	0.5, 0.75	0.5, 0.75	LAYER	LAYER	WOOD
" "	1930						GUNITE
RELIEF	1910	140	0.5	1.5	11	108	CONCRETE
CHRISTIAN VALLEY	1916	37	0.5	1.25	3	3	MASONRY
MAIN STRAWBERRY	1916	142	1.2	1.3	4	8	CONCRETE
BUCKS STORAGE	1928	128	1.4	1.5	3	6	"
SALT SPRINGS	1931	328	1.1 TO 1.4	1.4	15	15	"
LOWER BEAR RIVER NO. 1	1952	245	1.3	1.4	10	22	"
LOWER BEAR RIVER NO. 2	1952	150	1.0	1.3	10	17	"
WISHON	1958	290	1.0 TO 1.3	1.4	7.8		"
COURTRIGHT	1959	310	1.0 TO 1.3	1.4	7.8		"

* HEIGHT ABOVE FOUNDATION AT AXIS

TABLE 2.—PHYSICAL DATA FOR THE SALT SPRINGS AND LOWER BEAR RIVER DAMS

ITEM	SALT SPRINGS	LOWER BEAR RIVER NO. 1	LOWER BEAR RIVER NO. 2
YEAR COMPLETED	1931	1952	1952
STORAGE, ACRE FEET	142,000	49,000	
CREST ELEVATION	3958	5820	5820
HEIGHT AT AXIS, FT.	328	245	150
CREST LENGTH, FT.	1,300	960	865
SURFACE AREA OF FACE, SQ. FT.	380,000	190,000	90,000
SLOPES (HORIZ. TO 1 VERT.)			
UPSTREAM	1.1 TO 1.4	1.3	1.0
DOWNSTREAM	1.4	1.4	1.3
PLACED ROCK THICKNESS, FT.			
TOP	15	10	10
BOTTOM	15	21	17
QUANTITIES, CU. YD.			
EXCAVATION	312,000	10,000	138,000
DUMPED ROCK	2,744,000	900,000	349,000
PLACED ROCK	224,000	89,000	43,000
CONCRETE	45,000	23,000	11,500

TABLE 3.—TABLE OF CREST SETTLEMENT, IN FEET, FOR LOWER BEAR RIVER DAM NO. 1

STA.	HT.	*	YEARS					% OF HT.	
			1ST	2ND	3RD	4TH	4YRS	1YR	4YRS
1+40	110	V	.77	.02	.02	.05	.86	.70	.78
		H	.57	.00	.07	.00	.64	.52	.58
3+20	220	V	1.10	.03	.05	.05	1.23	.50	.56
		H	.74	.06	.07	-.01	.86	.34	.39
4+40	220	V	1.03	.03	.04	.06	1.16	.46	.53
		H	.86	.02	.07	.01	.96	.39	.44
5+60	150	V	.89	.02	.05	.02	.98	.59	.65
		H	.76	.03	.02	.01	.82	.51	.55
6+80	85	V	.23	.01	.02	.03	.29	.27	.34
		H	.22	.01	.03	.00	.26	.26	.31

* V = VERTICAL COMPONENT H = HORIZONTAL

of naturally compacted gravel and boulders were left in place. Abutments were washed of nominal and occasional surface overburden by hydraulic monitoring. The dam was constructed on clean, sound granite except in the streambed downstream from the axis.

Cutoff and Grouting.—A 5-foot wide cutoff trench varied in depth from 3 feet at the top to 15-20 feet at the bottom of the dam. Below points on the cutoff where the dam exceeds a 200-foot height (el. 3770) the 2-inch grout holes were 50 feet deep, and above they varied from 50 feet to 25 feet at the crest. Below el. 3770 holes were drilled and grouted at 6-foot centers, and above at 20-foot centers, intermediate holes being used if grout take exceeded 50 sacks. A total of 161 holes averaged 42 feet in length and 27.6 sacks of cement per hole.

Dumped Rockfill

Quarrying.—Of the 2,960,000 cubic yards in the dam 880,000 came from the spillway quarry and the balance from 3 quarries on the other abutment, all near the dam. Each quarry was worked from one level since the main hauling system was by rail. The faces of the completed quarries are vertical, 60 to 180 feet high and show the vertical drill holes. Rows of 6-inch churn drill holes on 18 to 22-foot centers were located 30 to 45 feet back from the face, and 2-inch drill holes at 8-foot centers were used for lifters at the quarry floor. The largest shot (Nov. 5, 1929), forty-four 158-foot main holes and 99 lifter holes loaded with 60 tons of powder, yielded 232,000 cubic yards of solid granite. The average powder factor was 0.60 lbs. per cubic yard of solid rock. This quarry method yielded many rocks that required secondary drilling and shooting to produce rock that could be handled. The compressive strength of the granite was 15,000 to 19,000 psi, as determined by tests on cubes and cylinders.

Grading of Rockfill.—The quarry method provided many large rocks, but the heavily loaded 6-inch holes also produced a substantial amount of fines. The secondary shooting produced large clean rock. The Bucyrus 120B (4 cu. yd.) shovels could handle 25-ton rocks and main quarry hauling was by 30 cu. yd. side dump and 20 cu. yd. end dump rail cars. Rock grading was from fines to 25-ton rocks, with the average large rock exceeding 3 tons. It was estimated that half the shovel loads were on the dipper teeth which indicates many large rocks. One shovel on two 8-1/2 hour shifts and a six-day week averaged 50,000 cu. yd. per month. The fill contains about 29% voids which represents a bulking factor of 41%. Field notes and photographs indicate that the rock coming from the quarry varied from large clean rock just after a quarry blast to smaller rocks with many fines when cleaning up the quarry. When "too many" loads of fines, on cleaning the quarry floor, were going into the dam, some loads were wasted. Total waste was less than 5%. Occasionally large rocks were dumped over visible areas of fines.

The photograph, Fig. 2, shows the 3910 lift and the 3800 lift coming together. The rock of the left abutment 3910 lift is observed to be much smaller than that of the 3800 lift. This was uniformly the case since the spillway quarry was equipped to handle rocks up to only 2 cu. yd. size, whereas the right abutment quarries could handle 8 cu. yd. and larger rocks.

Lifts.—Fig. 1 illustrates the lifts used in construction. The first fill in the 3670 and 3710 lifts was to permit an early start on the cutoff and placed rock and the lifts were of minimum top width. The three main lifts, Fig. 1, were each for the full thickness of dam. The main and most important lift of the dam was 185 feet high. Since the natural slope was about 1.30 to 1.35:1, it was necessary to construct berms beyond the 1.4:1 specified slope. Surface cleanup on the lifts was not good. The important surface was that at el. 3800 and the field engineer's report reads, "The spillway quarry 3910 lift is overlapping with no 3800 cleanup . . . an attempt is now being made to roughen the top of the 3800 lift by shooting holes 20 ft. apart and afterward sluicing fines into the larger voids at the bottom of these holes . . . a 3-inch low velocity jet moves the fines but holes 3 to 4 feet deep and 8 feet in diameter quickly silt up . . . a higher velocity jet would be desirable but is not possible . . . as the higher lifts overlapped the 3800 level it became dangerous to work on the cleanup". This is significant since the subsequent horizontal crushing failures all took place at el. 3800. The 185-foot dump from the 3800-foot level would have permitted considerable fines to accumulate in the top zone of the lift.

The 165-foot main lift was certainly a great benefit to Salt Springs Dam. The many large rock of 10 to 25 tons do considerable compaction as they slide or plow down the dumped slope. The height of lifts in the upper 199 feet or so of the dam are less important since pressures due to water and rock loading are both low.

Sluicing.—The superior performance of the Lower Bear River Dams over Salt Springs occasioned a thorough review of the Salt Springs construction records, particularly regarding sluicing. It is definite that Salt Springs sluicing was not comparable to present practice. The first 750,000 cubic yards were dumped between Feb. 1928 and July 1929 with only a 100 gpm-350-foot head-centrifugal pump and 4 and 2-inch pipes and hoses on the job.

Dumping had been on the order of 80,000 cu. yds. per month and sluice water less than 20,000 cu. yds. per month and not available at all points of dumping. A 300-gpm pump was added in July 1929. Sluicing was still recognized to be inadequate and on October 1, 1929, with 1,100,000 cu. yds. already in the dam, one 1200 and one 1500 gpm pump was installed along with 8 and 4-inch lines, and sluicing began in earnest. The 3800 lift of 1,500,000 cu. yds. was completed in Jan. 1930 and was thus only nominally sluiced. Though the last 1,900,000 cu. yds. of rock was dumped with a substantial amount of water, the method of application was not that which would be used today. The application of water was by letting hoses pour onto the top of the lift at or near the point of dumping. There were no jets to remove pockets of dirt and spalls and even with adequate volume of water the method of application would not permit the water to be fully effective. The fact that sluicing of the fill below el. 3800 was very nominal is important in that it may explain why the dam settled relatively greater below el. 3800 than in the lower part of Lower Bear River Dam No. 1.

Settlement During Dumping.—As dumping of rock progresses, the rock going down the slope seems to pull the previously dumped fill downward. This movement is illustrated by Figs. 1 and 4. On Aug. 12, 1929, points 1-6 were set on the top of the 3800 lift. As the fill progressed, points 7-9 were added and intermittent readings were taken. The settlement of points 6-9 is the full settlement which took place during the dumping from the 165-foot lift, except for the two weeks that the rock was from 0 to 25 feet from the edge of the berm. Point 6 settled 1.9 feet vertically plus an estimated 1.0 feet during the first two weeks for a total of about 3 feet, or about 2% for the 165-foot lift. This is perhaps more than should occur with a fill of similar rock sluiced in accordance with present practice. The rate and amount of the

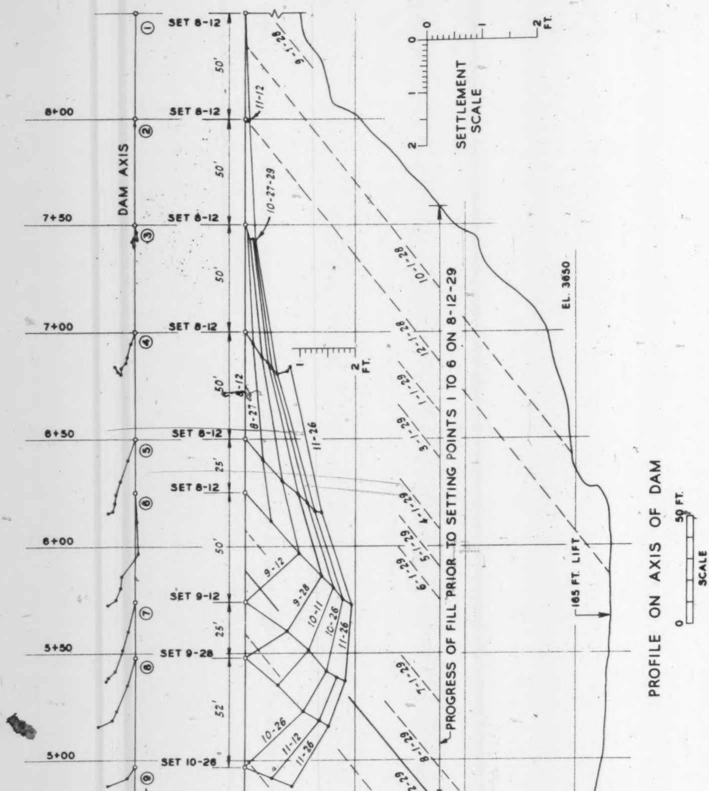

MOVEMENT IN PLAN OF POINTS 1 TO 9
POINT NOS.
DAM AXIS
FT. 1.0 0.5
8+00
7+50
7+00
6+50
6+00
5+50
5+00
4+50
4+00
3+50
① ② ③ ④ ⑤ ⑥ ⑦ ⑧ ⑨
SET 8-12
SET 9-12
SET 9-28
SET 10-28
50'
25'
52'
11-12
10-27-29
8-12
8-27
11-26
9-12
9-28
10-11
10-26
9-1-28
10-1-28
12-1-28
1-1-29
3-1-29
4-1-29
5-1-29
6-1-29
7-1-29
8-1-29
8-12-29
9-1-29
10-5-29
11-26-29
TOP OF EL. 3800 LIFT
FILL FROM EL. 3910
EL. 3750
EL. 3700
EL. 3650
PROGRESS OF FILL PRIOR TO SETTING POINTS 1 TO 6 ON 8-12-29
165 FT. LIFT
SETTLEMENT SCALE
0 1 2 FT.
PROFILE ON AXIS OF DAM
0 50 FT.
SCALE

FIG. 4.—SETTLEMENT DURING DUMPING OF MAIN LIFT - SALT SPRINGS DAM

movement diminishes with increasing distance back from the berm where dumping is taking place. The "closing fill" does not get much settlement. This settlement during dumping is one evidence of the benefit in compaction from high lifts. Such movement does not affect the placed rock or concrete face since it occurs so early in the construction.

The downstream movement of the points is believed due to the influence of the 3710 lift. The maximum section of Fig. 1 shows the relative location of the points and the 3710 lift. On November 5, 1929, the largest quarry blast, 58 tons of powder producing 232,000 cubic yards of solid rock, was set off. The settlement points were carefully measured before and after and no movement was observed to be caused by the blast.

Placed Rock

Placed rock was 15 feet thick normal to the face for the full section, Fig. 1. Rocks generally were placed with a flat face in the plane of the surface to minimize excess concrete over the design thickness. The average production rate for the total 224,000 cubic yards placed was 50 cubic yards per 9-hour shift of a crawler crane with crew of 8 men. The cranes operated on the surface of placed rock which is inconvenient and more costly than operating on "face lifts" as used at Wishon and Courtright, where production rate is about 2-1/2 times faster.

Masonry.—Along the cutoff the placed rock was bedded in concrete for the upstream 10 feet of thickness and several feet above the cutoff joint. This may have contributed to causing the abutment cracks and has not been done on subsequent company dams. The rigid masonry would decrease the flexibility near the cutoff joint, and cause a relatively abrupt transition of settlement from the unyielding masonry to the more yielding placed rock and fill.

Concrete Face

The concrete contained a minimum of 5.0 sacks of cement per cu. yd.; water-cement ratio was specified as 1.0 and ranged from 0.95 to 1.10; and 28-day strengths generally ranged from 2700 to 3200 psi. The concrete contained no admixtures and crushed aggregate of 2-1/2 in. maximum size was used. It was placed by chutes; was "wetter" than would be used today; and the field reports complain of segregation. Honeycombing occurred in spots due to mortar running into voids in the placed rock. However, its condition after 27 years of service in a severe climate attest that it was good concrete for that time.

Four cores were taken in 1946 to determine the condition of the concrete, 2 at el. 3712 and 2 at el. 3780. Chemical analysis showed little, if any, loss of cement by leaching. The concrete tested only 22% greater than the 28-day strength after 16 years and the modulus was only 2,680,000 psi.

Performance of Salt Springs Dam

The performance(21,22) has been very satisfactory in all respects. The face repairs have been nominal in cost in comparison with the appreciable cost saving in the use of this type of dam at this site, and have never involved safety. However, it is important to keep in mind that the performance to be reviewed, in terms of settlement, face cracks, leakage and maintenance, would be much better for a dam constructed at the same site with the experience and knowledge that is available today.

Settlement

Settlement During Construction.—Salt Springs Dam settled more during construction than the subsequent P. G. and E. Co. dams which were thoroughly sluiced in accordance with current practice. The settlement during the main lift dumping, as previously discussed, occurs so early in construction that it has little or no effect on the face. Evidence of additional settlement is:

(a) The upstream face was laid out to a concave slope up the face with 1.45:1 at the bottom and 1.1:1 at the top. It was attempted to place the placed rock to the surface design points on the profile. Settlement during construction required a steeper slope to be laid up to get back to the design location and finally to be at or near the design location on reaching the crest. Concrete was poured to the design thickness over the placed rock and not to the design surface in order to conserve concrete. Cross sections showing the relative locations of the design face and of the concreted face before filling the reservoir, show that at maximum sections these faces on completion of the dam were 3 feet apart in the central portion and 2 feet apart at the crest. At 150-foot high sections on each abutment, the two faces were 1 foot to 0.5 foot apart. Since the placed rock was placed to or slightly below the design lines, these values represent settlement between the time of placing the face rocks and completion of the dam.

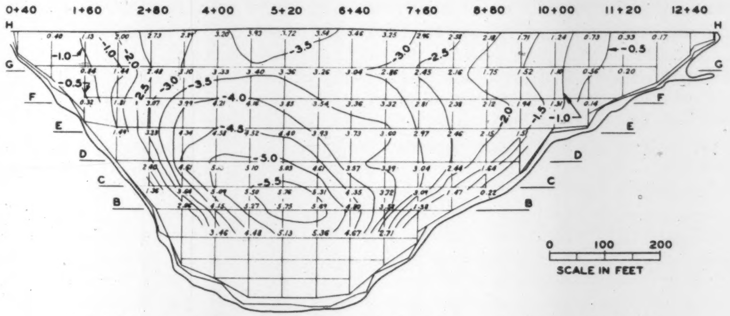
(b) Fig. 6 shows the latter part of the movement mentioned in (a) for Points A through F, which is movement that took place after the concrete face was poured at each point. Point D moved 1.4 feet normal to the face during the nine months of dam construction above Point D.

(c) Some surface placed rocks were observed to crush during construction.

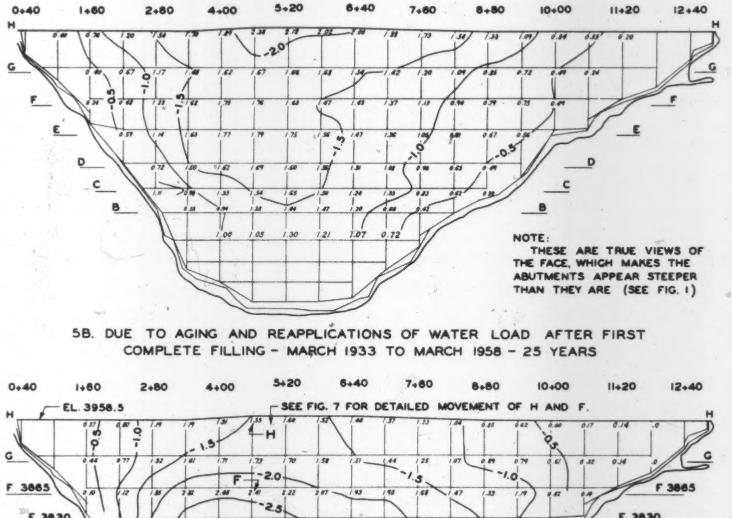
(d) The many abutment cracks that appeared before the filling of the reservoir, Fig. 6, were due to settlement during construction and after the concrete face was poured.

Settlement Contours.—A fundamental and particularly favorable characteristic of the impervious face rockfill dam is that movement of the face is "essentially" normal to the face. Stresses and joint movements occur in the plane of the face due to deviations from a line normal to the face, but the major component is normal. The contours of the resultant and "essentially" normal settlement are drawn on Fig. 5 to show the settlement: (1) due to first complete water loading; (2) due to 25 years of aging after the first complete filling; and (3) the total due to first filling and the 27-year life of the dam.

Fig. 5A shows the movement due to the first complete filling. This took place during two years since in 1931, the first year, California experienced a historic dry year and the reservoir did not fill. The shape is similar to the Lower Bear River Dam No. 1 contour of Fig. 14 with maximum movement at about four-tenths of the height. However, the maximum settlement at Salt Springs is 2.7 (4.32 + 1.60) times the crest settlement whereas at Lower Bear River Dam No. 1 it was only 1.4 (2.05 + 1.45) times the crest settlement. This is a significant difference and it is considered that the very high movement in the lower part of Salt Springs is due to the inadequate sluicing below el. 3800. The high settlements in the lower part of Salt Springs caused high lateral movements in the upper portion of the dam, and caused abutment

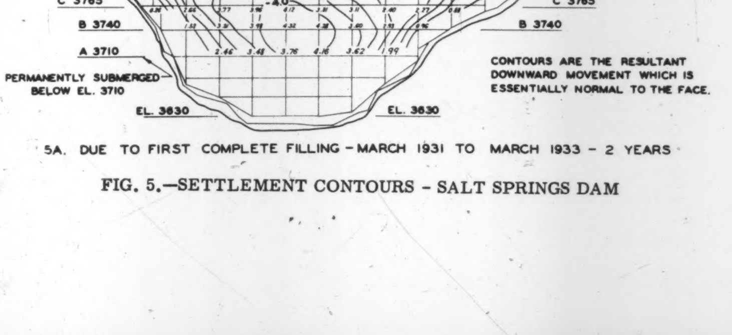

0+40
1+60
2+80
4+00
5+20
6+40
7+60
8+80
10+00
11+20
12+40
H
G
F
E
D
C
B
-1.0
-0.5
-1.0
-2.0
-2.5
-3.0
-3.5
-4.0
-4.5
-5.0
-5.5
-3.5
-3.0
-2.5
-2.0
-1.5
-1.0
-0.5
0
100
200
SCALE IN FEET

5C. TOTAL SETTLEMENT SINCE COMPLETION OF DAM - SUM OF 5A AND 5B
CONTOURS - MARCH 1931 TO MARCH 1958 - 27 YEARS.


0+40
1+60
2+80
4+00
5+20
6+40
7+60
8+80
10+00
11+20
12+40
H
G
F
E
D
C
B
-0.5
-1.0
-1.5
-2.0
-1.5
-1.0
-0.5
1.00
1.05
1.30
1.21
1.07
0.72

NOTE:
THESE ARE TRUE VIEWS OF THE FACE, WHICH MAKES THE ABUTMENTS APPEAR STEEPER THAN THEY ARE (SEE FIG. 1)

5B. DUE TO AGING AND REAPPLICATIONS OF WATER LOAD AFTER FIRST
COMPLETE FILLING - MARCH 1933 TO MARCH 1958 - 25 YEARS


0+40
1+60
2+80
4+00
5+20
6+40
7+60
8+80
10+00
11+20
12+40
EL. 3958.5
SEE FIG. 7 FOR DETAILED MOVEMENT OF H AND F.
H
G
F 3865
E 3830
D 3795
C 3765
B 3740
A 3710
PERMANENTLY SUBMERGED BELOW EL. 3710
EL. 3630
EL. 3630
H
F
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
-1.5
-1.0
-0.5
2.46
3.48
3.76
4.16
3.62
1.99

CONTOURS ARE THE RESULTANT DOWNWARD MOVEMENT WHICH IS ESSENTIALLY NORMAL TO THE FACE.

5A. DUE TO FIRST COMPLETE FILLING - MARCH 1931 TO MARCH 1933 - 2 YEARS

FIG. 5.—SETTLEMENT CONTOURS - SALT SPRINGS DAM

cracks in addition to those that occurred during construction. It is important to keep in mind that the large resultant settlements in the lower portion of the dam, and their unfavorable consequences of lateral movement and cracks, are excessive primarily because of the inadequate sluicing.

Fig. 5A shows that the combination of water pressure and height of rockfill that give maximum movement is located near four-tenths the height of dam. All other points may be considered to move more than would be caused by the water load at each point due to the drag from the area of maximum movement. For example, there is no water pressure at the crest which at the center moved 1.60 feet.

Fig. 5B shows that the settlement due to the aging of the fill and due to the reapplication of water load each year is different from that due to the first filling. The movement seems to be due more to the weight of the rockfill than due to the reapplication of water load. The high movement at the crest is probably due to the fact that the rock points in the upper portion were never very heavily loaded and aging is more effective on these less stabilized rock contacts. It is very favorable that the repeated reapplication of the substantial water load does not appear to cause significant movement of the rock.

Fig. 5C shows the contours of face movement for the 27 years since the beginning of the first filling, and is the sum of Contours 2A and 2B. Except in an area along the abutments, this settlement has caused no cracks. The reference plane for the contours is the position of the face on 3-9-31, just prior to the first filling of the reservoir, as shown on Fig. 3. The concrete slabs in the lower and central portion of the dam have moved even more than the contours show, as is shown by the location of points when set, Fig. 6.

Settlement at Maximum Section.—Fig. 6 shows the record of vertical and horizontal movement for the maximum section. The fact that the reservoir did not fill the first year made it possible to obtain information on the full face due to the partial filling. Some comments on Fig. 6 are:

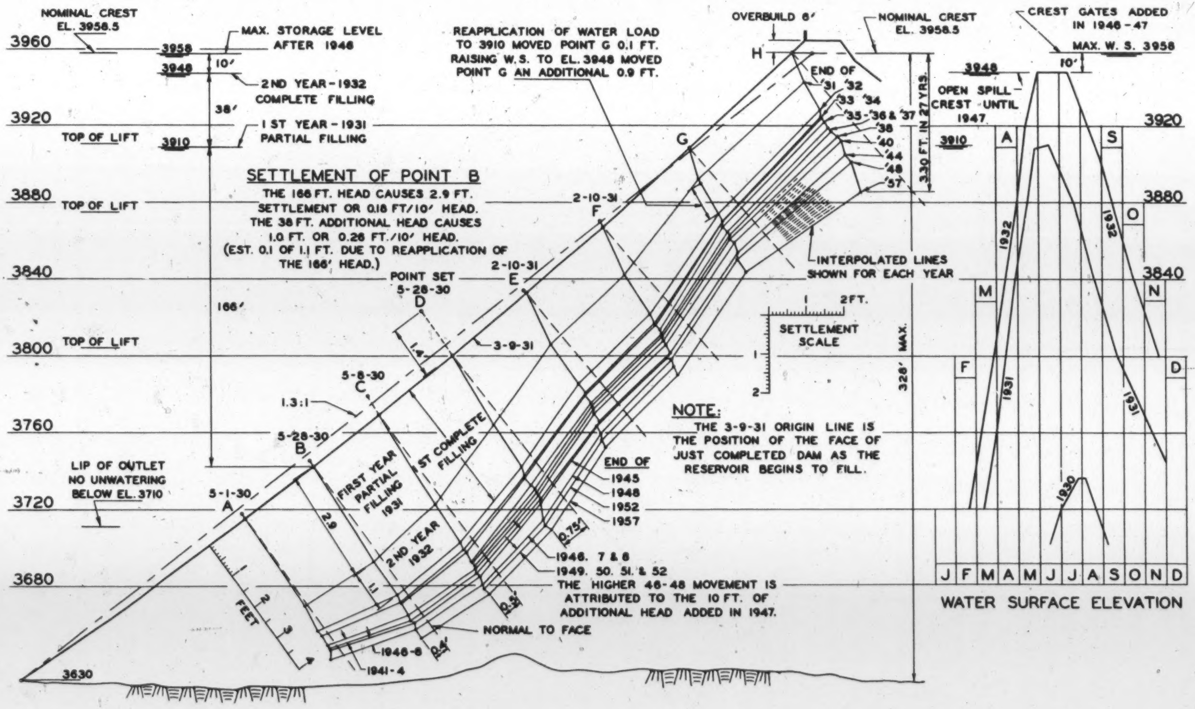
(a) Point B moved 2.9 feet due to the water surface rising to el. 3910, and 1.0 feet due to the application of the next 38 feet of head, estimating 0.1 feet of the 1.1 feet was due to reapplication of the 166 feet of head. This indicates an increasing rate of settlement as head increases. This is equivalent to saying that the coefficient of settlement increased as head changed from 166 feet to 204 feet.

(b) Point G moved 0.1 feet as the reservoir refilled to 3910 and another 0.9 feet due to the 38 feet of additional head.

(c) Settlement is down from normal. The slabs DE, DC, CB, and BA were all in compression in a line down the face as determined by the shortening. The horizontal joints were cold joints so that this shortening caused compressive stresses. For example, the 44-foot slab length CB has shortened 0.1 foot without crushing. Slabs HG and GH are under tension in a line up the face and horizontal joints G and F have opened to relieve that tension.

(d) Vertical crest settlement. 3.6 feet of the 6-foot overbuild has been used up in the 27 years to 1958.

(e) As also shown by Figs. 5 and 7, the maximum rate of settlement after the first filling is at the crest and not in the lower portion of the dam. This is favorable, since it indicates that the crushing of rock points in the lower portion of the dam, due to the first application of water load, leaves a rock mass capable of resisting repeated application of that



SETTLEMENT OF POINT B

THE 166 FT. HEAD CAUSES 2.9 FT. SETTLEMENT OR 0.18 FT/10′ HEAD. THE 38 FT. ADDITIONAL HEAD CAUSES 1.0 FT. OR 0.26 FT./10′ HEAD. (EST. 0.1 OF 1.1 FT. DUE TO REAPPLICATION OF THE 166′ HEAD.)

REAPPLICATION OF WATER LOAD TO 3910 MOVED POINT G 0.1 FT. RAISING W.S. TO EL. 3948 MOVED POINT G AN ADDITIONAL 0.9 FT.

NOTE:
THE 3-9-31 ORIGIN LINE IS THE POSITION OF THE FACE OF JUST COMPLETED DAM AS THE RESERVOIR BEGINS TO FILL.

THE HIGHER 46-48 MOVEMENT IS ATTRIBUTED TO THE 10 FT. OF ADDITIONAL HEAD ADDED IN 1947.

NOMINAL CREST EL. 3958.5
3960
3920
3880
3840
3800
3760
3720
3680
3630
TOP OF LIFT
TOP OF LIFT
TOP OF LIFT
3958
3948
3910
10′
38′
166′
MAX. STORAGE LEVEL AFTER 1948
2ND YEAR - 1932 COMPLETE FILLING
1 ST YEAR - 1931 PARTIAL FILLING
LIP OF OUTLET NO UNWATERING BELOW EL. 3710
POINT SET 5-28-30
5-1-30
A
5-28-30
B
5-8-30
C
D
2-10-31
E
2-10-31
F
G
H
3-9-31
1.3:1
1.4
2.9
1.1
FIRST YEAR PARTIAL FILLING 1931
1 ST COMPLETE FILLING
2ND YEAR 1932
FEET
1
2
3
1946-8
1941-4
0.4′
0.5′
0.75′
NORMAL TO FACE
1946. 7 & 8
1949. 50. 51. & 52
END OF
1945
1948
1952
1957
OVERBUILD 6′
NOMINAL CREST EL. 3958.5
END OF
'31 '32
'33 '34
'35-'36 & '37
'38
'40
'44
'48
'57
330 FT. IN 27 YRS.
326′ MAX.
INTERPOLATED LINES SHOWN FOR EACH YEAR
SETTLEMENT SCALE
1
2FT.
1
2
CREST GATES ADDED IN 1946-47
MAX. W. S. 3958
10′
3948
3910
OPEN SPILL CREST UNTIL 1947
3920
3880
3840
1930
1931
1932
J F M A M J J A S O N D
WATER SURFACE ELEVATION

FIG. 6.—SETTLEMENT AT MAXIMUM SECTION - STATION 4 + 60 - SALT SPRINGS DAM

heavy load with nominal additional settlement. The rock points in the upper portion of the dam, that have not been loaded as heavily by rock or water loading, settle at a higher rate in aging than does the more heavily loaded rock points.

(f) After the first filling, the settlement is increasingly down from normal to the face.

(g) The addition of crest gates in 1946 added 10 feet of head that influenced settlement for the 3-year interval from the end of 1945 to the end of 1948. It is favorable that the 10 feet of additional head caused nominal movement. This seems inconsistent with the comments of (a) above, except that 15 years of aging settlement took place before this additional 10 feet was added to the maximum storage level.

Settlement of Points H and F.—These two points on the maximum section, Fig. 5A, are used to illustrate the first three years of settlement in detail. The settlement is plotted, Fig. 7, with movement in feet as abscissa and at the elevation of the water surface when the reading was taken. The number of months between readings is shown on the line between points. The progressive first three years of movement is summarized on Fig. 7 for the vertical settlement of Point H.

These charts show movement for the first three years in greater detail than Figs. 4 and 5 and are typical points. Both points exhibit similar characteristics. During the several months of drawdown (approximately 80 feet) after the first, second and third filling, Point H and similar adjacent points showed vertical rebound of 0.01, 0.02 feet or zero settlement. During the remaining 120 feet of drawdown, taking place during 6 or 7 months, Point H settled vertically some 0.16, 0.8, and 0.4 for the first, second and third year. The horizontal movement was quite different. Point H continued to move horizontally during the first several months of drawdown (approximately 80 feet) and then would rebound horizontally 0.03 to 0.04 feet while it was settling vertically. Horizontal rebound of Point F was 0.04, 0.09 and 0.01 feet, which is greater than vertical rebound. It appears that permanent horizontal settlement takes place while reservoir is substantially filled and a horizontal force is present. However, the vertical settlement goes on while reservoir is empty due to the weight of rock. This is one indication of the influence of reservoir operation on the pattern of settlement. The reservoir operation of 1932, as shown on Fig. 6, is typical of Salt Springs operation.

The bar graphs of settlement, Fig. 7, show the rate of settlement after the large movement due to the first complete filling. The crest, Point H, has settled vertically after the first two years at a rate that has decreased from 0.10 ft per yr to 0.02 ft per yr in 29 yr. The horizontal component has decreased in twenty-five years from 0.07 to 0.025 feet per year. At Point F, located 92 feet lower than the crest, the vertical rate of movement has decreased from 0.076 to 0.044 foot per year, but has been constant for the last thirteen years at 0.044 foot per year, a little less than the 0.055 feet per year for Point H. On the other hand, horizontal movement of Point F has been about the same as that of Point H in the last ten years, and is very small at about one-quarter inch per year.

Lateral Settlement.—The term lateral settlement refers to the horizontal movement in the plane of the face of the dam. Such movement causes opening and closing of the vertical joints and/or stresses across the face slabs. Fig. 8 presents lateral measurement readings for four significant dates: (1) just prior to first filling: (2) after first year partial filling and unwatering;

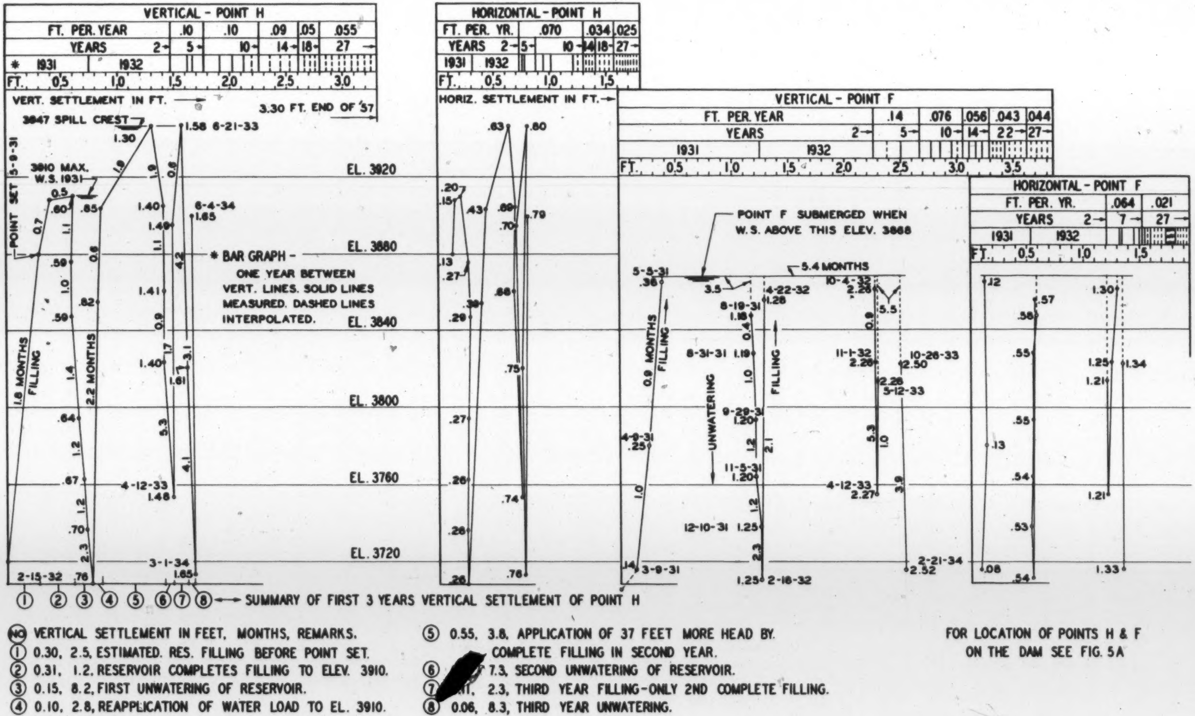
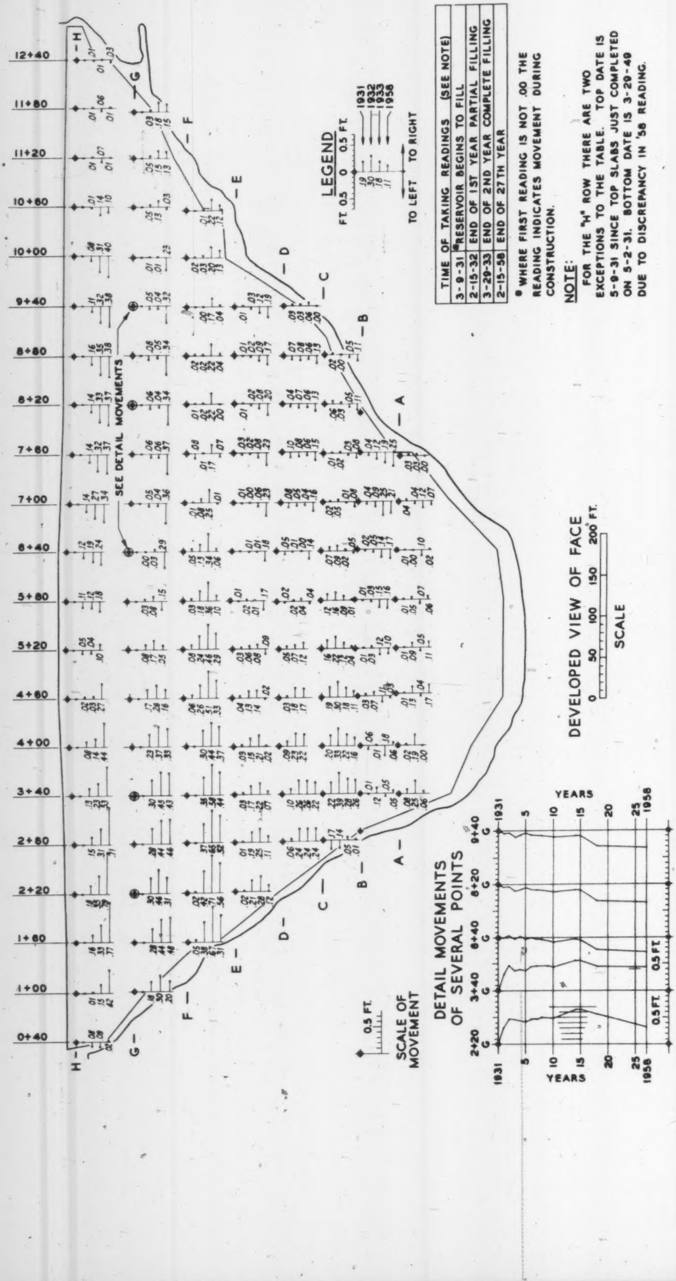


FIG. 7.—SETTLEMENT OF POINTS "H" AND "F" OF MAXIMUM SECTION - STATION 4 + 60 - SALT SPRINGS DAM



TIME OF TAKING READINGS (SEE NOTE)	
3-9-31	*RESERVOIR BEGINS TO FILL
2-15-32	END OF 1ST YEAR PARTIAL FILLING
3-29-33	END OF 2ND YEAR COMPLETE FILLING
2-15-58	END OF 27TH YEAR

* WHERE FIRST READING IS NOT .00 THE READING INDICATES MOVEMENT DURING CONSTRUCTION.

NOTE:

FOR THE "H" ROW THERE ARE TWO EXCEPTIONS TO THE TABLE. TOP DATE IS 5-9-31 SINCE TOP SLABS JUST COMPLETED ON 5-2-31. BOTTOM DATE IS 3-29-49 DUE TO DISCREPANCY IN '58 READING.

FIG. 8.—LATERAL FACE MOVEMENT - SALT SPRINGS DAM

(3) after second year complete filling; and (4) after twenty-seven years. The detailed data on five points of the G-line are also shown. Some observations are:

(a) The major lateral movement took place during the high and rapid initial movements due to the partial and complete fillings of the first two years.
(b) Movement is away from the abutments and toward the center of the dam, with maximum movement in the upper portion of the dam.
(c) The movements are greatest on the left abutment.
(d) The increase in movement between 1946 and 1949, as noted on detail movement of G-Line points, is considered to be due to the raising of storage water surface by 10 feet in 1947.
(e) In the past approximately ten years most of the face has moved toward the left abutment which is the primary direction of dumping during construction of the dam. This tendency in the aging of the dam is different from the lateral movement associated with the initial water loading.

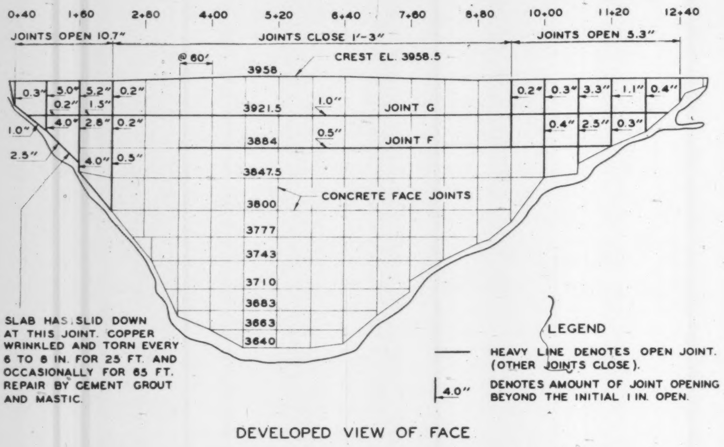
The water load applied to the upstream face, which is convex upstream in plan, tends to push the rockfill from the abutments and toward the center. At the crest at the center of the dam the upstream curvature is represented by a 13.6-foot middle ordinate in the 1300-foot crest length. A second factor considered to cause this movement is the differential adjustment in the rockfill due to the high normal movements shown on the contour 5A of Fig. 5. As water pressure causes significant downstream and downward differential movement, a lateral readjustment of rock points must take place. It appears that this lateral readjustment of rock contacts, along with the active pressure within the rockfill and the high percentage of voids, causes the rocks to move toward the center of the dam. In addition to these factors the basic vertical settlement should tend to develop a component toward the center due to the sloping abutments.

It seems that the high initial lateral movements are associated with the high normal settlement in the lower and central portion of the dam. This apparent fact is of interest and value and if true, means that these high lateral movements would be much less for a presently constructed dam that does not have the high resultant settlements of Salt Springs Dam.

The face measurements give a record of the surface lateral movement of the face slabs, but the movement in the fill must have been still greater since the face resists movement. As vertical joints close in the central portion of the dam, the slabs accept high stresses and the rock points beneath the slab can adjust to permit movement in the fill without movement in the face.

Horizontal Joint Movement.—The horizontal joints were cold joints with copper waterstops. They could open, but could not close to relieve compression stresses due to movement of the underlying rockfill or due to temperature rise. Fig. 9 shows that the two upper joints, F and G, opened. The remaining joints remained closed. This indicates that the two upper rows of slabs are in tension in a line up the face, and below Joint F, in compression.

Vertical Joint Movement.—The joints that slope up the face in a vertical plane are called vertical joints to differentiate from the sloping joints that parallel the abutment. The several vertical joints near each abutment opened and those approaching and at the center of the dam closed, Fig. 9. All vertical joints were constructed one inch open and the openings show the movement beyond that one inch. The larger openings on the left abutment and the sliding

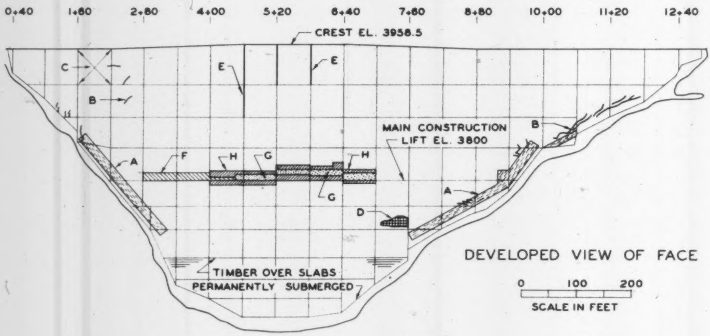

0+40
1+60
2+80
4+00
5+20
6+40
7+60
8+80
10+00
11+20
12+40
JOINTS OPEN 10.7"
JOINTS CLOSE 1'-3"
JOINTS OPEN 5.3"
@ 60'
CREST EL. 3958.5
3958
0.3"
5.0"
5.2"
0.2"
0.2"
1.5"
1.0"
4.0"
2.8"
0.2"
2.5"
4.0"
0.5"
3921.5
1.0"
JOINT G
3884
0.5"
JOINT F
3847.5
0.2"
0.3"
3.3"
1.1"
0.4"
0.4"
2.5"
0.3"
CONCRETE FACE JOINTS
3800
3777
3743
3710
3683
3663
3640
LEGEND
4.0"
DEVELOPED VIEW OF FACE

SLAB HAS SLID DOWN AT THIS JOINT. COPPER WRINKLED AND TORN EVERY 6 TO 8 IN. FOR 25 FT. AND OCCASIONALLY FOR 65 FT. REPAIR BY CEMENT GROUT AND MASTIC.

HEAVY LINE DENOTES OPEN JOINT. (OTHER JOINTS CLOSE).

DENOTES AMOUNT OF JOINT OPENING BEYOND THE INITIAL 1 IN. OPEN.

FIG. 9.—FACE JOINT MOVEMENTS—SALT SPRING DAM


0+40
1+60
2+80
4+00
5+20
6+40
7+60
8+80
10+00
11+20
12+40
CREST EL. 3958.5
C
B
E
E
A
F
H
G
H
G
MAIN CONSTRUCTION LIFT EL. 3800
B
A
D
TIMBER OVER SLABS PERMANENTLY SUBMERGED
DEVELOPED VIEW OF FACE
0
100
200
SCALE IN FEET

A - BEFORE FILLING CRACKS 6 IN. TO 2 FT. APART GENERALLY PARALLEL TO ABUTMENT AND 1/8 TO 1/4 IN. OPEN ON SURFACE.

B - 1931 AND 1932 - CRACKS LOCATED AFTER EMPTYING RESERVOIR AFTER FIRST COMPLETE FILLING.

C - 1931 AND 1932 - SLAB FULL OF HAIRLINE CRACKS IN ALL DIRECTIONS.

D - 1938 - SPALLING OF AREA ABOVE THE PLANE OF THE REINFORCING.

E - JOINTS CRUSHED - REPLACED WITH 3 IN. OPEN JOINTS.

F-G-H - CRUSHING OF SLAB IN HORIZOTAL LINE ACROSS SLAB.

F - MAR. 1941 - 2.7 SLABS.

G - MAR. 1944 - 4.3 SLABS.

H - MAR. 1948 - 5.0 SLABS ABOVE AND BELOW THE NEW CONCRETE OF 1944.

FIG. 10.—RECORD OF ALL FACE CRACKS - SALT SPRINGS DAM

along the sloping joint are attributed to the construction methods. The fill dumped from the spillway quarry contained excessive fines and sluicing was not used initially. The construction notes and Fig. 2 show the thin fill dumped from elevation 3910 to be inferior to the fill from right abutment quarries. The central joints have closed and compressive stresses reached a point where three of them have crushed. The opening of joints has caused no trouble. The vertical joint movement is a manifestation of lateral settlement.

Face Cracks

All cracks that have occurred in the face of the dam are shown on Fig. 10. The great majority of slabs have experienced no cracking. It is considered that all cracks that did occur could be avoided in a dam constructed with today's knowledge. Comments on the various types of face failures, using the letter designations of Fig. 10, are:

A. Cracks during construction. These appeared before filling of the reservoir in an area 10-30 feet from the abutment. The cracks were generally parallel to the abutment and about 1/16 inch wide, with maximum being 1/2 inch. Initial repair was by chipping and using expansive mortar. With time, further joint opening and frost action resulted in deteriorated concrete zones. These areas were chipped out and replaced with new concrete, the last of such repair being made in 1958.

B. Cracks during filling. The few cracks that developed were repaired by grout and/or mastic.

C. Hairline cracks. Only one slab developed hairline cracks. This occured during filling. The slab was in tension in both directions.

D. Spalling of surface. One area spalled off above the plane of the reinforcing.

E. Crushing of vertical joints. Eight of the central joints closed the one-inch constructed opening completely and three crushed. They were replaced by 3-inch open joints which at the top are completely closed in 1958. The central vertical joints are resisting lateral movement and some may crush in the future.

F., G. and H. Crushing of horizontal joint. The three failures all took place in March with low reservoir and exceptionally hot weather. The settlement caused compression in a line down the face in this area and apparently the additional temperature stress caused failure. The time of failure was convenient since repairs could immediately be made and no leakage resulted. The joint that failed is at elevation 3800, the level of the main construction lift which was not well cleaned up, and the top of an inadequately sluiced lift. The last failure took place in 1948, upon the first unwatering after raising the water level by 10 feet.

Leakage.—Leakage has been produced primarily by cracks "A" of Fig. 10, by honeycomb pockets in the concrete face, and by several open joints. Points of leakage may be located by observing clumps of pine needles on the face of the dam. As long as leakage did not approach or exceed the required release, face repairs were not made.

Table 4 shows the leakage in cfs for the last 10 years from the 380,000 square foot surface and the abutments of Salt Springs Dam.

It is observed that there is little leakage from the lower 212 feet of the face.

TABLE 4

Water Level	Depth Feet	Leakage At Different Reservoir Levels										
		1947	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957
3958	320	13	20	16	19	8	12	14	16	16	11	16
3900	262	6	9	7	7	3	3	6	8	8	4	8
3850	212	2	4	3	3	2	2	3	3	3	3	3

Maintenance

Crack and Frost Damage Repair.—Maintenance has consisted of repair to the cracks and crushed areas described above and of local small concrete patches where frost action had taken place. The concrete poured in 1931 had occasional porous or honeycomb areas on the surface. Frost action enlarges such zones and also the concrete adjacent to cracks. The patches arrest the frost action and have been satisfactory. Frost action has been worse on the right abutment side of the face which is exposed to winter sun, and more cycles of freezing and thawing. The left abutment is in the shade during the winter.

Cost.—In 27 years of life of the dam, the maintenance cost has averaged $8,000 per year. This compared with a $2,000,000 estimated saving, in 1931 dollars, in using a rockfill rather than a concrete dam. Maintenance has not affected operation and the cost has been nominal.

Present Condition.—A thorough inspection of the face was made in February 1958. The original concrete appears to be sound, except in small local areas affected by frost action. Nominal patching of such local spots should arrest the deterioration due to frost action. Water chemical action on the face has removed very little surface mortar and presents no problem. Nominal future patching should extend the life of the slab into the indefinite future. The dam is in excellent condition.

The Lower Bear River Dams

The Lower Bear River Dams No. 1 and No. 2(23,28) are located on the Bear River just below the Upper Bear River rockfill dam, are only a few miles from Salt Springs Dam, and are near the Carson Pass Highway about 40 miles out of Jackson, California. They were completed in 1952.

The 245-foot No. 1 dam is the main dam and the 150-foot No. 2 dam is a wing dam, the two being separated by a granite knoll, Fig. 11 and 19. The reservoir stores 49,000 acre feet for use in a power drop of one mile in four P. G. and E. Company plants.

In connection with the Lower Bear River dams, the purpose of this paper is to thoroughly present the performance data and more briefly present design and construction data. Comparisons will be made with Salt Springs Dam. The two Lower Bear River Dams will be discussed separately.

Design of Lower Bear River Dam No. 1

The site was exposed massive granite requiring no streambed excavation and nominal excavation of loose material from pockets on the abutments. The concrete face rockfill dam was selected on the basis of lowest estimated cost and adaptability to the site and to the operation of the reservoir.

The outlet works are illustrated and described in references (23) and (28). The spillway is a double side channel spillway located on the knoll between the two dams and discharges in an unlined channel, Fig. 11. The design capacity is 12,000 cfs. This discharge is passed with a depth of 4.6 feet over the open crest weir, and 3.4 feet of freeboard on the 4.0-foot high coping wall.

The main design features are as shown on the section, Fig. 11, and the joint arrangement is as shown on Fig. 14. The face is moderately curved in plan, and is the shape of a cylinder of 4350-foot radius tilted on a 1.3:1 slope. At the crest the center arches 14 feet upstream and at 0.4 height only 6 feet. This arching is just enough to give a favorable appearance after settlement and to give a tendency toward compression across the face rather than tension. The moderate compressive stress and closing of joints is preferred.

The 1.3:1 upstream face slope requires greater quantities than the 1.33:1 to 1.1:1 slope for the top 245 feet of Salt Springs Dam. The exposed rock foundation and the narrow canyon made the greater quantities nominal and it was thought that lower bid prices due to convenience of building the face to the natural slope would offset the cost associated with the increased quantities. Observations during construction indicated that this was not the case, and the subsequently designed Wishon and Courtright face slopes are variable and are steeper than that of Salt Springs.

The Lower Bear River designs are significant as a transition between the early Salt Springs design and the recent Wishon and Courtright dam designs. The remaining significant features will be reviewed in comparison with Salt Springs.

Comparison with Salt Springs Design

Improvements.—Though the face troubles at Salt Springs were not serious, it was desired to obtain a more watertight dam at Lower Bear River. Also, the minimum reservoir operating level is 120 feet above the base of the dam, and unwatering the reservoir would require a shutdown of the plant and a loss of 2200 acre feet of dead storage. With the knowledge gained from Salt Springs, it was considered that a more watertight face could be obtained. Improvements to minimize settlement, face cracks and leakage are considered to be:

(1) Sluicing: High pressure sluicing, with water to rock ratio of 3 to 1 by volume, was specified to provide a fill that would give minimum settlement.
(2) Hinge Slabs: The 15-foot slabs along the abutment were provided to minimize or prevent cracks in the slabs near the abutments.
(3) Soft Horizontal Joints: To release compression that might develop in a line down the face, the horizontal joints contained one-half inch thick redwood boards in addition to the U-shaped copper waterstop, Fig. 17B.
(4) Lift Surface Cleanup: The lift surface was specified to be scarified, sluiced or otherwise prepared to obtain a rough exposed rock surface for contact with the succeeding lift.
(5) Two-Inch Open Joints: To prevent the crushing of vertical joints near the top and central portion of the dam, joints in that area were constructed 2 inches open, Fig. 17.
(6) Additional Horizontal Joints: The horizontal joint spacing is the same

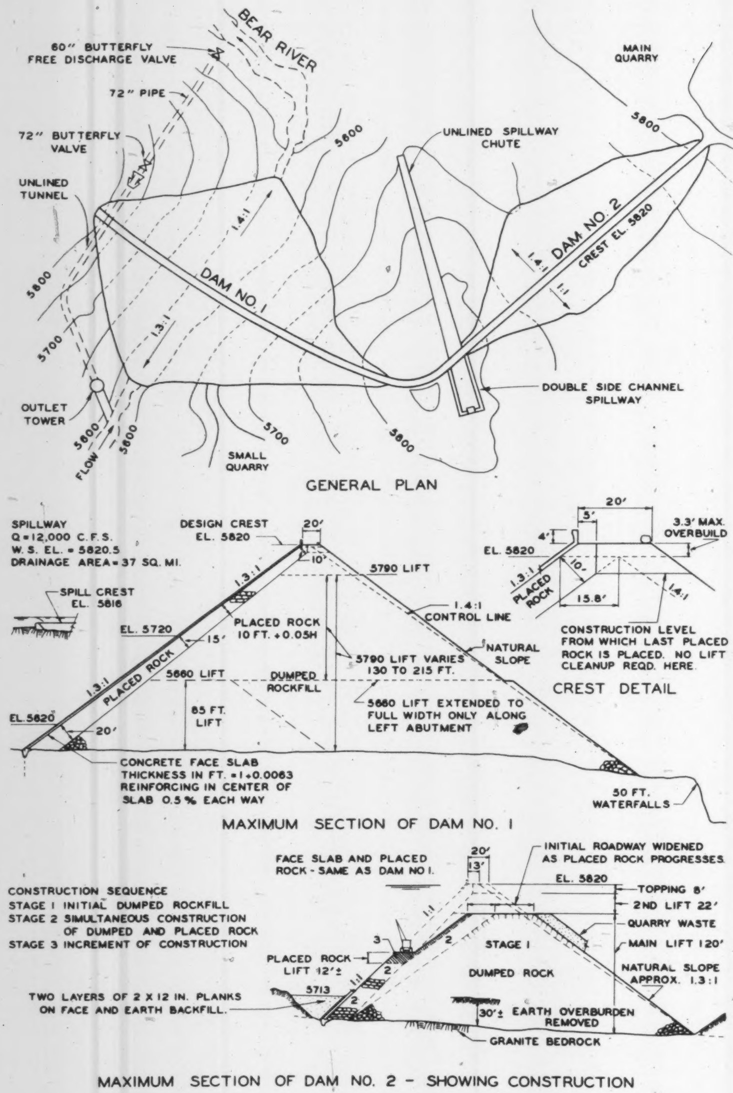

BEAR RIVER
60" BUTTERFLY FREE DISCHARGE VALVE
72" PIPE
72" BUTTERFLY VALVE
UNLINED TUNNEL
MAIN QUARRY
5800
UNLINED SPILLWAY CHUTE
5800
1.4:1
DAM NO. 2
CREST EL. 5820
1.4:1
1:1
5800
DAM NO. 1
1.3:1
5700
OUTLET TOWER
5800
FLOW
5800
5700
SMALL QUARRY
5800
DOUBLE SIDE CHANNEL SPILLWAY
GENERAL PLAN
SPILLWAY
Q = 12,000 C.F.S.
W. S. EL. = 5820.5
DRAINAGE AREA = 37 SQ. MI.
SPILL CREST EL. 5816
DESIGN CREST EL. 5820
20'
10'
1.3:1
5790 LIFT
1.4:1 CONTROL LINE
PLACED ROCK 10 FT. + 0.05H
EL. 5720
15'
NATURAL SLOPE
5790 LIFT VARIES 130 TO 215 FT.
1.3:1
PLACED ROCK
5660 LIFT
DUMPED ROCKFILL
85 FT. LIFT
5660 LIFT EXTENDED TO FULL WIDTH ONLY ALONG LEFT ABUTMENT
EL. 5620
20'
CONCRETE FACE SLAB
THICKNESS IN FT. = 1 + 0.0063
REINFORCING IN CENTER OF SLAB 0.5 % EACH WAY
50 FT. WATERFALLS
MAXIMUM SECTION OF DAM NO. 1
20'
5'
3.3' MAX. OVERBUILD
4'
EL. 5820
10'
1.3:1
PLACED ROCK
15.8'
1.4:1
CONSTRUCTION LEVEL FROM WHICH LAST PLACED ROCK IS PLACED. NO LIFT CLEANUP REQD. HERE.
CREST DETAIL
FACE SLAB AND PLACED ROCK - SAME AS DAM NO I.
20'
13'
INITIAL ROADWAY WIDENED AS PLACED ROCK PROGRESSES.
EL. 5820
TOPPING 8'
2ND LIFT 22'
QUARRY WASTE
MAIN LIFT 120'
CONSTRUCTION SEQUENCE
STAGE 1 INITIAL DUMPED ROCKFILL
STAGE 2 SIMULTANEOUS CONSTRUCTION OF DUMPED AND PLACED ROCK
STAGE 3 INCREMENT OF CONSTRUCTION
1:1
3
2
STAGE 1
DUMPED ROCK
PLACED ROCK LIFT 12'±
2
NATURAL SLOPE APPROX. 1.3:1
5713
1:1
2
TWO LAYERS OF 2 X 12 IN. PLANKS ON FACE AND EARTH BACKFILL.
30'± EARTH OVERBURDEN REMOVED
GRANITE BEDROCK
MAXIMUM SECTION OF DAM NO. 2 - SHOWING CONSTRUCTION

FIG. 11.—GENERAL PLAN AND SECTIONS - LOWER BEAR RIVER DAMS

as at Salt Springs except for two additional soft joints in the central part of the dam.

Economies.—The economies over Salt Springs design were:

(1) Placed Rock: The thickness at the top was changed from 15 to 10 feet and 5 feet were added for each 100 feet of height, Fig. 11. Because the face area of the top half of the dam is much greater than in the lower half, this resulted in less volume of placed rock. It put the thicker placed rock in the area of greatest movement and in the area that is not unwatered in normal operation.

(2) Cut-Off Trench: The cut-off trench was decreased from 5 to 3 feet in width and from 15 to 7.5 feet deep at the base of the dam. The specified depth was 3 feet for the top of both dams.

Identical Features.—Several design features were left unchanged and are as follows:

(1) Slab Thickness and Reinforcing. The theoretical thickness varies from 1 foot to 3 feet in 300 feet of height and is reinforced in the center with 0.5% reinforcing each way, Figs. 1 and 11.

(2) Downstream Slope. The 1.4:1 layout line was used at Salt Springs in case the natural dumped slope was that flat. It was steeper and overbuilt berms resulted, Fig. 1. Though the natural slope is considered adequate the 1.4:1 control slope was used at Lower Bear River, Fig. 11. It gives a somewhat flatter than natural slope and makes the layout determinate regardless of the actual dumped slopes.

(3) One-Inch Open Vertical Joints. Except for the upper portion of the central joints being constructed 2 inch open, the rest are 1 inch open as at Salt Springs. Though it may be predicted that certain joints will open, the 1 inch is considered desirable to permit assured articulation of the slabs.

(4) Joint Rib Keyways. The design was the same but the rectangular vertical keys were changed to triangular shape during the job to make the placing of rock easier.

Construction of Lower Bear River Dam No. 1

The job was awarded to Utah Construction Company in July 1950 and completed in October 1952. The diversion tunnel, opening of quarry, construction of haul roads, clearing and excavation were accomplished in the remaining four months of the 1950 construction season. Due to snow at this 6000-foot elevation in the Sierras, the construction season is about six months, May through October. The actual dam construction of both dams took one year, six months in 1951, and six in 1952.

A detailed and illustrated description of construction is presented in references (24), (25), (26) and (27). Photographs of construction are presented in Figs. 12, 13 and 18, and completed dam in Fig. 19. Table 2 gives a tabulation of physical data.

Foundation and Cutoff

The foundation is solid granite. No streambed excavation was required and pockets of overburden on the abutments were removed by sluicing.

Grouting was in 25-foot deep holes at 10-foot centers with every fifth hole 50 feet deep. The 50-foot holes were core holes and water tested. Data for the cutoff grouting for the 2510-foot length of cutoff for both No. 1 and No. 2 dams is: one hole every 9.7 feet; 4.3 feet of drilling per lineal foot of cutoff; 3.4 sacks of cement per foot of cutoff; 85% of cement in holes taking more than 50 sacks; and 41 of 258 holes took more than 2 sacks of cement. In the distance between Station 1 + 80 to 6 + 80 on Dam No. 1 no hole took more than 1 sack of cement.

Dumped Rockfill

Quarry.—The location and number of quarries was left to the Contractor, the only requirement being that it must be at least 150 feet from the cutoff of the dam if the quarry floor were below crest elevation. The rock for both dams was obtained from one main quarry and one small toe quarry, Fig. 11 and 18. The "coyote" quarry method was used.(24,25) 1,260,000 cu. yds. of solid rock from the Main Quarry were obtained in 9 blasts with 0.60 lbs. of powder per cu. yd. and 460 cu. yds. of solid rock per foot of 3.5 by 5.0-foot coyote hole. The largest shot used 89 tons of powder and yielded 307,000 cu. yds. of solid rock. The coyote method produced a substantial amount of fines near the coyote holes and many large rocks that required secondary drilling.

Composition of Rockfill.—The rock is gray, fine-grained grano-diorite that has a compressive strength of 15,000 to 20,000 psi. Quarry run rock of predominantly large size, 1 - 10 tons, was specified. Fines of 4 inch and less in size were specified to be less than 5% by weight. As constructed, more than 50% of the rock is estimated to be 1 to 20-ton size and many loads with more than 5% fines were used in the fill. Where fines were dumped over large clean rock they were not rejected. Including overburden, a weathered and decomposed zone and the cleanup of fines on the quarry floor, the waste was roughly about 15%. Fig. 12 shows the character of the rockfill. Note that though large rocks segregate near the toe, there are many that remain in the upper portion of the lift.

Lifts.—Specifications called for not more than 3 lifts plus a topping lift for the 245-foot high dam and for lift surface cleanup of fines. Two main lifts were used, Fig. 11, with part of the dam constructed in one main lift. The 5660 lift of minimum top width, was to give an early start to the construction of placed rock. It is of minimum top width to provide earliest completion, and minimum surface cleanup. Also, it is of minimum top width to let some of the next lift be of greater height. However, in order to keep all shovels and trucks busy, a portion of the 5660 lift was extended to the downstream slope along the left abutment. Fine material cleaned up from lift surfaces was cast over the downstream face and gives the dam the appearance of being of smaller rock than is the case. The upstream face dumped slope on nine sections of 45 to 85-foot height averaged 1.28:1 and the horizontal distances for 1 vertical were: 1.25, 1.27, 1.21, 1.27, 1.26, 1.33, 1.34, 1.33, 1.24. A dumped slope of large rock is quite irregular. The upstream slope of 20 sections of 40 to 105-foot height on Dam No. 2 averaged 1.24:1 and were in the range of 1.20:1 and 1.32:1. These upstream slopes were of selected loads of the larger rocks of 2 to 15 tons. The average downstream slope for 16 sections of 60 to 120-foot height on Dam No. 2 was 1.31:1 and ranged from 1.26 to 1.40. The 1.31:1 average slope was for rockfill of smaller and more broadly graded rock than the upstream slopes.

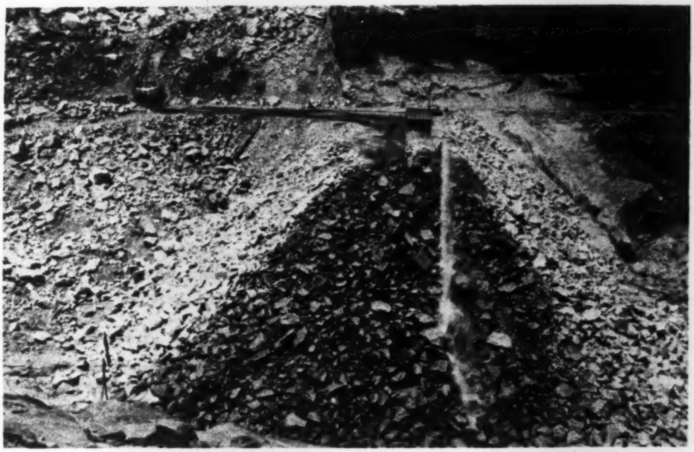


FIG. 12.—DUMPING ROCK ON 120-FT LIFT OF LOWER BEAR RIVER DAM NO. 2

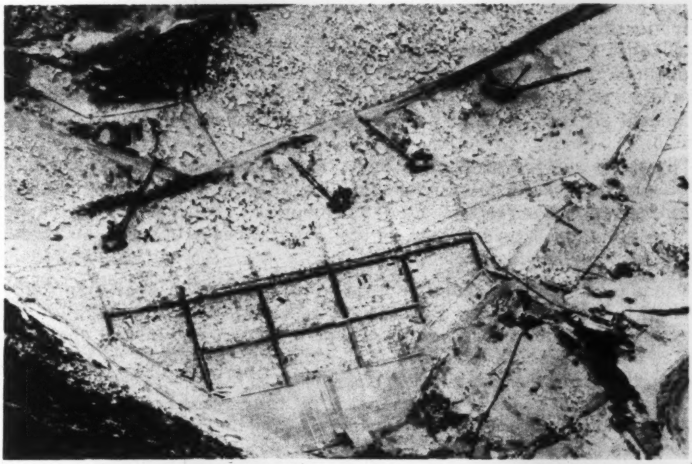


FIG. 13.—UPSTREAM FACE SHOWING DUMPED ROCK, PLACED ROCK AND CONCRETE FACE CONSTRUCTION OF LOWER BEAR RIVER DAM NO. 1

Sluicing.—Sluicing was at a volume ratio of water to rock of 3:1. The ratio is rather arbitrary and if the rock is relatively clean 2:1 would probably be quite adequate. A cost study showed that for the 2:1 ratio the cost was 4.6 cents per cu. yd. and 75% of the cost was the labor cost of the monitor operator. For 3:1 the labor is the same and the increased pumping energy and piping cost only increased the unit cost by 0.4 cents. Therefore, the 3:1 ratio was adopted.

Four monitors were used with 2-1/2 or 3-inch nozzles and 80 to 90 psi pressure at the nozzle. A pumping plant of 3 electric 10 x 12-4210 gpm-275-foot head Ingersoll Rand pumps were used. Normal operation was with 2 pumps and 3 monitors. Piping began with a 12-inch line and branches to 8 or 6 inch lines with 25-foot lengths of 6 or 8-inch heavy rubber hose leading to the monitors. Monitor discharge ranged from 2 to 4 cfs. At Salt Springs, with no pressure, the sluicing water softened or moved dirt and quarry dust but did not excavate pockets of spalls as did the jet sluicing at Lower Bear River.

Settlement During Construction.—During the 6 months of 1951 construction season 70% of the dumped rock and 14% of the placed rock was constructed on Dam No. 1. The end of the first 3 months of the 1952 season saw the dumped rock nearly finished and the placed rock just reaching its peak rate of 15,000 cu. yds. per month. Except for several slabs poured at the base of the dam in 1951 to serve as the diversion dam, all slabs were poured in late 1952. The placed rock was placed to the theoretical shape of the face and significant movement after placing was not observed. The concrete was poured to the theoretical points on the face. If movement of placed rock had taken place it was intended to pour concrete to required thickness and not to theoretical layout points.

On Lines T. U and V settlement points were set on October 8, 1952 while upper slabs were being poured. On November 28, 1.7 months later, readings were taken as the dam was being snowed in for the winter. The maximum settlement for the 1.7 month period was 0.095 on the U-line at Station 3 + 80, Fig. 16.

The small movements during construction are in sharp contrast with the greater movements of Salt Springs, and the difference in sluicing method is considered to be the reason.

Placed Rock

The placed rock was specified to be of predominantly large size rock and to obtain maximum rock to rock contact with voids kept to a minimum. Voids were chinked. Rocks were set down to give forward and lateral contact as well as bottom contact. Keyways were formed in the placed rock for the joint ribs, in the same way as at Salt Springs.(22)

The placing of rocks(24,25) was from crawler cranes of 1-1/2 to 3-1/2 cu. yd. size, and as many as 7 cranes were working on Dam No. 1 at one time. The cranes operated from the top of the placed rock, Fig. 13, and moved across the dam placing a layer of 10 to 12-foot height. This required a reasonably level surface which slows down the work and was still sufficiently rough that timber pads were often used to walk the cranes. Selected loads of large clean rock were dumped on the upstream face to provide a supply of rock to the cranes, such that removal of the large rocks would not leave an inferior rockfill surface of fines and small rock adjacent to the placed rock. Handling was by cable slings and required about 6 men per crane.

The 112,000 cu. yds. required for both dams took 2980 crane shifts which gives an average of 38 cu. yds. per 8-hour crane shift. At the peak of the job 29,000 cu. yds. were placed in August, 1952 at rates of 42 and 47 cu. yds. per crane shift for Dams No. 1 and No. 2 respectively. The 38 cu. yds. per crane shift compares with 50 for Salt Springs, 100 for Bowman and 50 to 100 for some other dams. Much consideration was given to the Specifications and procedures for the Company's recent Wishon and Courtright Dams, and rates there average about 135 and peak at 200.

Concrete Face.—Concrete was specified to be 3000 psi concrete and to contain 5 to 6% entrained air. The concrete face(27) was poured largely by Pumpcrete, but also by crane and bucket where crane access was possible. Specially designed steel forms of 2'-6" by 7'-0" dimension were continually moved ahead of the concrete on channel whalers that were fastened to the placed rock at 7 ft. spacing. The forms were stripped while the concrete was still green and the surface given a wood float. Initially, the steel forms were left in place until needed and the surface was observed to contain many air bubbles. The procedure was changed to remove them promptly and give the surface a wood float finish. A hard dense surface is desirable to resist the frost action. The concrete work progressed very rapidly, all but the four bottom slabs being poured in August, September and early October of 1952. Due to the surface steps and irregularities of the placed rock, the average slab thickness exceeds the design thickness. This excess thickness averaged 16 inches for the dam.

Performance of Lower Bear River Dam No. 1

The service record of this dam has been very satisfactory to date, and covers a period of five years. Only two face cracks have developed, maximum leakage has been 3 cfs, and settlement has been low.

Settlement

During Construction.—Based on Salt Springs experience, it was expected that the placed rock face would settle as the dam was being constructed and that, in steepening the slope to place rock to the design lines, the shape up the face would be concave. The concrete was planned to be poured to design thickness and not to layout lines in order to avoid excess concrete and obtain the concave line. The movement was so nominal that the concrete face was poured to the design layout lines on a 1.3:1 slope. The numbers at the points on the U and V lines, Fig. 16, give the vertical settlement for 51 days between setting the bronze pins and completion of the dam on November 28, 1952 (40 days for T line). The maximum movement was 0.095 foot or 1.15 inches, which is very small.

Settlement Contours.—The reservoir is normally not drawn down below el. 5700, Fig. 15, but it was completely emptied in November 1954 which made it possible to obtain complete settlement measurements for the two-year period. These are shown as contours and as horizontal sections in Fig. 14 and as vertical sections in Fig. 16. The face has adjusted to the shape shown by the contours without cracking. Some comparisons with Salt Springs have been discussed.

It is interesting that the maximum settlement is toward the left and steeper abutment of both dams. Assuming a uniformly compacted rockfill, the point

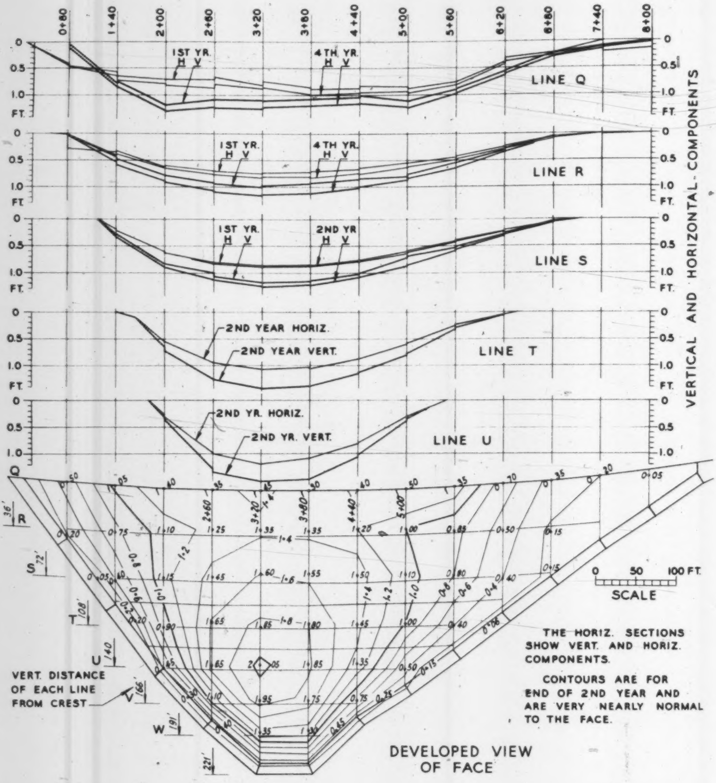

0+80
1+40
2+00
2+60
3+20
3+80
4+40
5+00
5+60
6+20
6+80
7+40
8+00
1ST YR. H V
4TH YR. H V
LINE Q
LINE R
2ND YR H V
LINE S
2ND YEAR HORIZ.
2ND YEAR VERT.
LINE T
2ND YR. HORIZ.
2ND YR. VERT.
LINE U
0
0.5
1.0
FT.
VERTICAL AND HORIZONTAL - COMPONENTS
Q
R 36'
S 72'
T 108'
U 140
V 166'
W 191'
221'
VERT. DISTANCE OF EACH LINE FROM CREST
0 50 100 FT.
SCALE
THE HORIZ. SECTIONS SHOW VERT. AND HORIZ. COMPONENTS.
CONTOURS ARE FOR END OF 2ND YEAR AND ARE VERY NEARLY NORMAL TO THE FACE.
DEVELOPED VIEW OF FACE

FIG. 14.—SETTLEMENT CONTOURS - LOWER BEAR RIVER DAM NO. 1

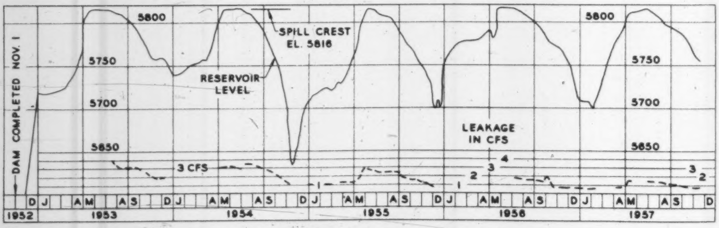
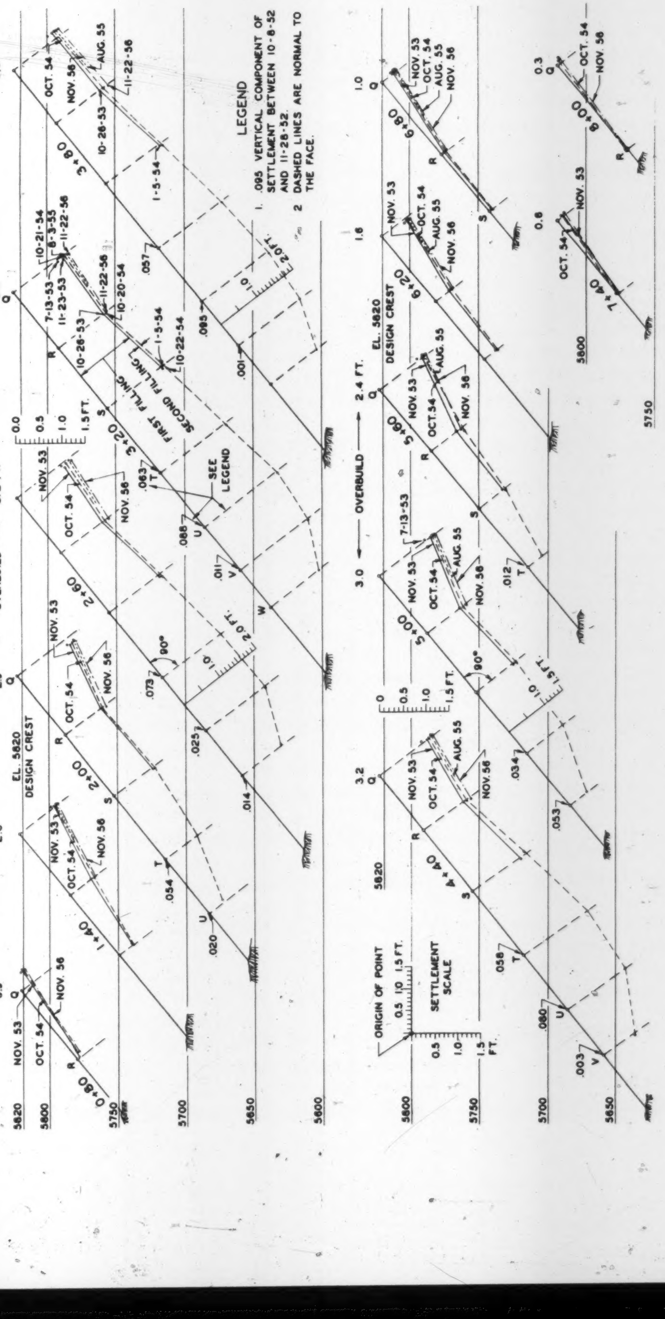


FIG. 15.—RESERVOIR OPERATION AND DAM LEAKAGE - LOWER BEAR RIVER RESERVOIR


5820
5800
5750
5700
5650
5600
0.9
Q
R
0+80
NOV. 53
OCT. 54
NOV. 56
2.0
1+40
T
U
.054
.020
EL. 5820
DESIGN CREST
2.9
2+00
S
.025
.014
.073
90°
1.0
2.0 FT.
OVERBUILD
3.2 FT.
2+60
.088
.011
V
W
.063
SEE LEGEND
0.0
0.5
1.0
1.5 FT.
3.3
3+20
FIRST FILLING
SECOND FILLING
7-13-53
11-23-53
10-21-54
8-3-55
11-22-56
10-20-54
10-26-53
1-5-54
10-22-54
.057
.095
.001
3+80
AUG. 55
11-22-56
10-26-53
1-5-54
ORIGIN OF POINT
SETTLEMENT SCALE
3.2
4+40
.058
.080
.003
.034
.053
3.0
5+00
7-13-53
.012
2.4 FT.
5+60
1.6
6+20
1.0
6+80
0.8
7+40
0.3
8+00
5800
5750

LEGEND

1. .095 VERTICAL COMPONENT OF SETTLEMENT BETWEEN 10-8-52 AND 11-28-52.
2. DASHED LINES ARE NORMAL TO THE FACE.

FIG. 16.—SECTIONS OF DAM SHOWING SETTLEMENT - LOWER BEAR RIVER DAM NO. 1

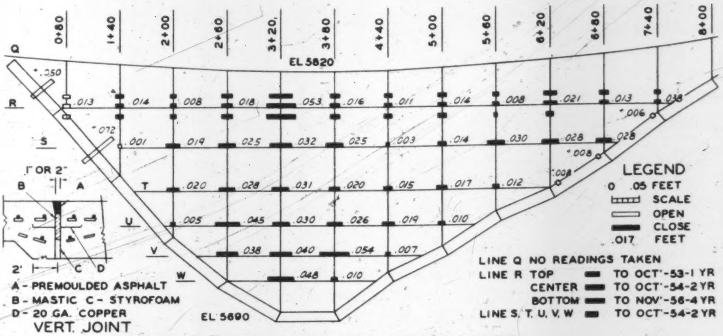
of maximum settlement should be toward the flatter abutment. The steep abutment should resist the drag of settlement, from the high movements of the maximum section, more than the flat abutment. The tendency to higher settlements near the left abutment of Dam No. 1 is considered to be due to the construction of all lifts from right to left abutment. At Salt Springs it is considered to be due to the main lifts going from right to left abutment and also due to the inferior fill from the left abutment spillway quarry. Of course the maximum differential settlements are near the steep abutment.

Settlement Sections.—The settlement sections, Fig. 16, and Table 3, show the progressive movement during four years. On July 13, 1953, six weeks after reservoir filled for the first time, crest readings were taken, Fig. 16. Five months later, after reservoir had lowered, Fig. 15, readings were again taken. The major settlement took place during the first filling. The movement during the last five months of the first year was not very great. That the face takes its major movement the first year and then moves very little is strikingly apparent and is very favorable in this type of dam. It means that if cracks are to occur, they should occur in the first year and after being repaired the dam might require little or no further scheduled unwatering or maintenance. That the movement is very close to normal to the face is also favorable since movement other than normal to the face causes stress in the slab and joint opening or closing. That the movement is essentially normal does not mean that the fill settles in the same way since the reinforced slab, that is interlocked to the large surface placed rocks, certainly resists any movement other than normal. Some readjustment of rock contacts in a zone underlying the face slab can permit the fill to move other than normal to the face without moving the face slab.

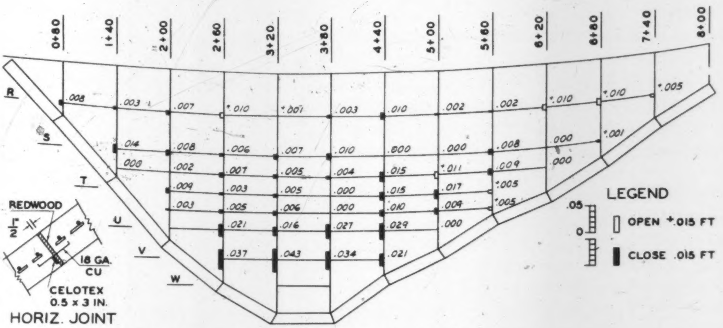
The significant settlement is that of the maximum section since its movement appears to drag the adjacent sections. This influence is seen by comparing the sections and tabulations of about equal height of Dam No. 1, Fig. 16 and Table 3, and Dam No. 2, Fig. 21. Comparison of sections of equal height on the same dam further shows the influence of maximum section on the abutments of different slopes. It is considered that expressing the vertical crest settlement or the horizontal crest settlement; i.e., deflection, as a per cent of height is only significant at the maximum section if the abutments are steep. Study of the tabulations of Table 3 and 21 show this.

The matter of dragging from the point of maximum movement also occurs within the section. The maximum settlement due to the first complete filling is at about 0.4 height. At the top there is no water load and for the 1.3:1 face dam the water load for the lower two-thirds of the face is transmitted to bedrock upstream from the axis, and does not directly affect the rock under the crest. The movement of the crest caused by water load is due to a drag and readjustment of the rocks and it appears reasonable that maximum settlement is not at the crest.

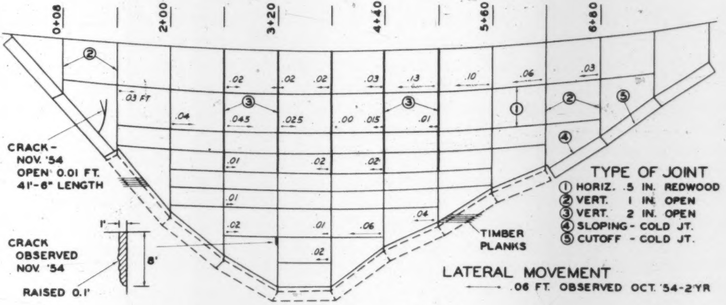
Vertical Joint Movement.—The vertical joint movements of Dam No. 1, Fig. 17A, are markedly less than those of Salt Springs, Fig. 9. The maximum joint closure is 5/8 inch at R 3 + 20 and V 3 + 80 where the joints were constructed 2 inch and 1 inch open, Fig. 17C. All joints except 0 + 80 close slightly. The vertical joint movement is very satisfactory and confirms the design practice of leaving all vertical joints a minimum of 1 inch open. Based on the three readings on the R-Line, the joint movement took place the first year.


0+80
1+40
2+00
2+60
3+20
3+80
4+40
5+00
5+60
6+20
6+80
7+40
8+00
Q
R
S
T
U
V
W
EL 5820
EL 5690
1" OR 2"
B
A
C
D
2'
A - PREMOULDED ASPHALT
B - MASTIC C - STYROFOAM
D - 20 GA. COPPER
VERT. JOINT
LEGEND
0 .05 FEET
SCALE
OPEN
CLOSE
.017 FEET
LINE Q NO READINGS TAKEN
LINE R TOP TO OCT'-53-1 YR
CENTER TO OCT'-54-2 YR
BOTTOM TO NOV'-56-4 YR
LINE S, T, U, V, W TO OCT'-54-2 YR

17A. OPENING AND CLOSING OF VERTICAL JOINTS


0+80
1+40
2+00
2+60
3+20
3+80
4+40
5+00
5+60
6+20
6+80
7+40
8+00
R
S
T
U
V
W
REDWOOD
1/2"
18 GA. CU
CELOTEX 0.5 x 3 IN.
HORIZ. JOINT
LEGEND
.05
0
OPEN +.015 FT
CLOSE .015 FT

17B. OPENING AND CLOSING OF HORIZONTAL JOINTS


0+80
2+00
3+20
4+40
5+60
6+80
.03 FT
CRACK - NOV. '54 OPEN 0.01 FT. 41'-6" LENGTH
CRACK OBSERVED NOV. '54
RAISED 0.1'
1"
8'
TIMBER PLANKS
TYPE OF JOINT
① HORIZ. .5 IN. REDWOOD
② VERT. 1 IN. OPEN
③ VERT. 2 IN. OPEN
④ SLOPING - COLD JT.
⑤ CUTOFF - COLD JT.
LATERAL MOVEMENT
.06 FT. OBSERVED OCT. '54-2 YR

17C. FACE CRACKS AND LATERAL MOVEMENT

FIG. 17.—MOVEMENT OF JOINTS, LATERAL MOVEMENT, AND FACE CRACKS - LOWER BEAR RIVER DAM NO. 1

Most of the joint closing is due to the resultant settlement which shortens the moderately arched face. Comparison of the net joint closing along lines R through W of Fig. 17A with the computed shortening is:

	R	S	T	U	V	W	
Close	.17	.15	.14	.14	.14	.07	feet
Shorten	.13	.13	.11	.08	.03	.01	feet

The difference is a measure of the net change in length of the total length of slabs due to horizontal stresses in the slabs. Because the central slabs are in compression and the abutments slabs in tension, the net change in length is not very great.

Horizontal Joint Movement.—Both vertical and horizontal joint movements are accurately determined by measuring between center punched 5/8-inch bronze pins on each side of the joint and near each corner of the slabs. The horizontal joints contain a 1/2-inch board of redwood and are termed soft joints. The wood filler rather than an extrudable filler was used to give some resistance to movements before yielding. Tests indicated that the redwood resisted 1000 psi when crushed to one-half thickness and 5000 psi when compressed to three-tenths its original thickness. The joint closings are all within an acceptable 0.03 feet except for several on the V-Line, Fig. 17B. The 0.043 closing at V 3 + 20 is associated with a local failure.

The top joint, R-Line, either closes very moderately or opens; all movements being less than one-eighth inch. The other horizontal joints close an increasing amount toward the bottom of the dam. Inspection of the contours and sections showing settlement make this seem reasonable. The upper portion of the face is supported by a substantial amount of rock that receives little water loading. The rock under the central and lower portion of the concrete face is pushed down without the restraint of the rock downstream from the axis and not loaded by the water. This tends to give greater compression toward the base of the dam and, in fact, tension in the top slabs.

Lateral Face Movement.—The lateral positions of points are determined by stationing along each line of points. The movements shown on Fig. 17C are in the range of 1/8 to 1/2 inch and are on the order of the accuracy of the chaining across the face of the dam. It is believed that the readings are not significant except to indicate that lateral movements are very small and that the lateral movements of the well sluiced Dam No. 1 rockfill are much less than for Salt Springs. It is repeated that the rockfill lateral movement is probably greater than measured on the slab due to the resistance of the slab and joints to movement. Measurements of the change in width of the 60-foot wide slabs show all abutment slabs to be in tension horizontally with elongations of 0.06 to 0.01 feet in 60 feet. Central slabs are in compression, having shortened generally 0.005 to 0.015. A shortening of 0.01 feet represents a compressive stress of about 700 psi.

Face Cracks and Maintenance.—Two cracks of different causes have occurred, Fig. 17C, in the five years of service. As shown on Fig. 17C, the abutment slabs, below the minimum operating level of 5700, are covered with two layers of 2 x 12-inch planks anchored to the concrete. Whether there are cracks in these hinge slabs is not known. The vertical crack near Station 1 + 40 is in a slab that has increased in width by about 0.04 feet and that slab is under tension in a horizontal line. It was filled and covered with rubberized asphalt.

The failure at V 3 + 20, Fig. 17C, is at a joint intersection where joints have closed 0.040 and 0.043, Fig. 17A and B. A 1 by 8-foot piece of concrete cracked and raised 0.10 feet. The 0.043 closure is of a joint that contains a 0.042 thickness of redwood. In order for a joint to close, the underlying concrete rib and some rock contacts must crush. With the high compression in both directions, the crushed rib could exert a large force to raise the edge of the slab. The broken zone was not removed to see what happened, but plastered with rubberized asphalt.

Leakage.—A V-notch leakage weir is located on massive exposed granite in the streambed below the dam. The leakage has never exceeded the required release and the maximum in 1957 was less than 2 cfs, Fig. 15.

Lower Bear River Dam No. 2

The No. 2 dam is a relatively small 150-foot high wing dam to the Main No. 1 dam, Fig. 11, 18 and 19. It is similar to Dam No. 1, but being of only 150-foot height some economies and simplifications in design were made. The No. 2 dam will be discussed in relation to Lower Bear River Dam No. 1.

Design

Features the Same as Dam No. 1.—The overall specifications were the same. The placed rock thickness, crest width, face reinforcing and cutoff design were alike. The vertical joints were the same, except that none were 2 inch open. The 1 inch joint was considered more than necessary for this low dam with gradual profile but was used as a practical minimum.

Features Different From Dam No. 1.—The primary difference in design is the upstream face slope of 1:1 as compared to 1.3:1 for Dam No. 1. For Dam No. 2, drill holes indicated that considerable earth overburden would have to be removed and minimum base thickness of dam was desirable. The 1:1 slope was adopted as being economic and suitable for the entire 150 feet of height of Dam No. 2. It was believed that the 1:1 face would be more difficult to construct. Observation of the simultaneous construction of the two dams of different face slopes indicated that unit costs should be about the same and placed rock was actually placed at a faster rate on Dam No. 2. Neither the "soft" horizontal joint nor the hinge slabs were considered necessary for the low height and gradual profile of this dam, and they were not used. Fewer horizontal joints were used and the crest alignment was straight.

Horizontal Joints.—The horizontal joints are essentially construction cold joints with copper waterstop and the reinforcing does not pass through the joints. At elevation 5713 the joint is useful in providing articulation of the face slab and also as a construction joint. The slabs below el. 4713 were covered with two layers of 2 x 12-inch planks which were used as forms and the hole below el. 5713 was then backfilled with earth, Fig. 11 and 20. In addition to providing a water seal the backfill gave construction road access for placing rock and concrete. The 5770 joint was for construction reasons.

Construction

Dumped Fill.—Dam No. 2 was constructed essentially in one lift, which became the haul road to Dam No. 1, Figs. 11 and 18. Quarry waste was dumped on the downstream face outside the downstream design slope of the dam to

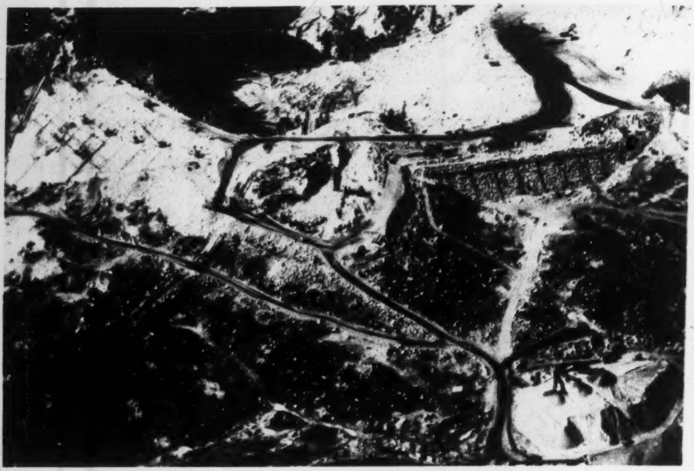


FIG. 18.—AERIAL VIEW DURING CONSTRUCTION LOWER BEAR RIVER DAMS NO. 1 AND 2

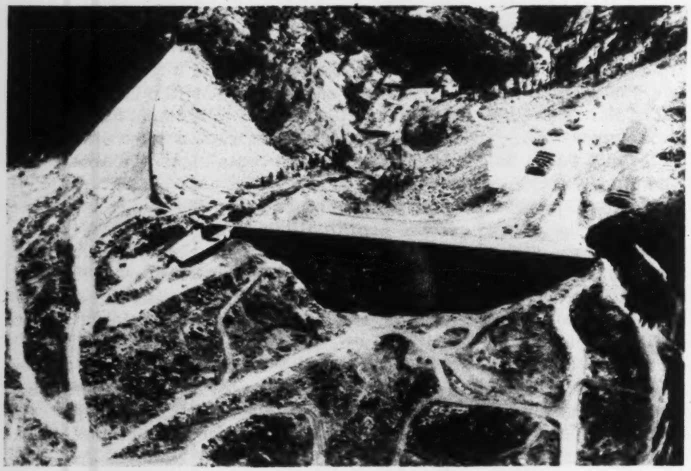


FIG. 19.—AERIAL VIEW OF COMPLETED DAMS PRIOR TO FIRST FILLING - LOWER BEAR RIVER DAMS NO. 1 AND 2

give a wider haul road and one that was available while the top small lifts were being constructed.

Placed Rock.—Fig. 11 shows the construction sequence. The lower placed rock of Stage 2 was placed from cranes operating from upstream of the cutoff or on the el. 5713 backfill. Stage 3 is an increment of construction where the placed rock was placed in lifts from a crane operating on the placed rock, dumped rock was then dumped and was caught by the placed rock.

Concrete.—Concrete was all placed by buckets handled by cranes. Cranes near the cutoff serviced the lower 80-foot slab and cranes on the crest serviced the top slab.

Performance

Dam No. 2 has had no face cracks, no leakage and no maintenance in its first five years of service. Settlement has been very moderate.

Settlement.—Settlement is shown by contours, section and tabulation on Fig. 20 and 21. The low settlement is associated with a dam, for which the Stage 1 main lift was used as a roadway to haul 800,000 cu. yds. of rock to Dam No. 1. Also, the main fill was completed in August 1951 and the water load applied one year and eight months later. The overfill of 176,000 cu. yds. of waste on a dam of 350,000 cu. yds. would have provided some resistance to horizontal deflection, but perhaps not to vertical settlement.

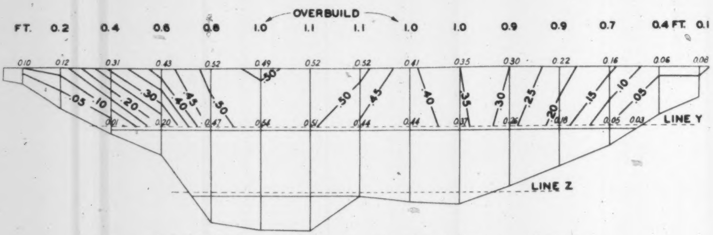
The movements as shown on the settlement contour drawings of Fig. 20 are much smaller than the movements of Salt Springs, but they have similar characteristics. The first year waterload settlement is greatest near four-tenths the height and the later aging settlement is greatest at the crest. Fig. 20B indicates that the aging settlement is greater near the left abutment. The dam being dumped from right to left abutment, the left abutment fill did not obtain the compaction of the drag of the dumped slope to the extent that the right abutment did. The crest settlement due to both water load and aging is about the same for Sections 13 + 35 and 18 + 15 even though the Section at Station 18 + 15 is twice the height. If the abutment profiles had been the same, more specific comments on the effect of the direction of dumping on the settlement could be made.

The sections and tabulation of Fig. 21 give a detailed record of the settlement during the first four years. The crest settlement of 0.21% for first year and total of 0.25% for four years, for the 140-foot high section at Station 15 + 75, is low.

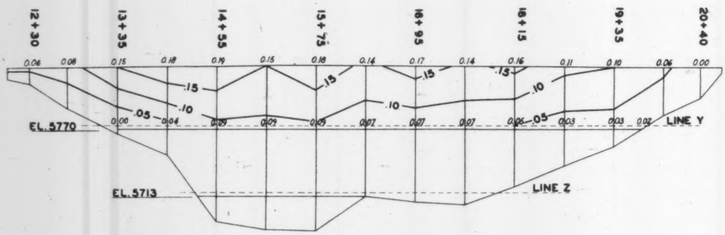
Joint Movement.—The measurements across the 1 inch open vertical joints are not shown because there has been zero opening or closing of the joints. As has been commented on in discussing the Dam No. 1 joints, it takes a large force to close a vertical joint and the temperature stresses alone have not been great enough to cause the necessary crushing in and below the joint ribs. It appears that this dam could have been faced with a continuous slab without experiencing any cracking.

CONCLUSION

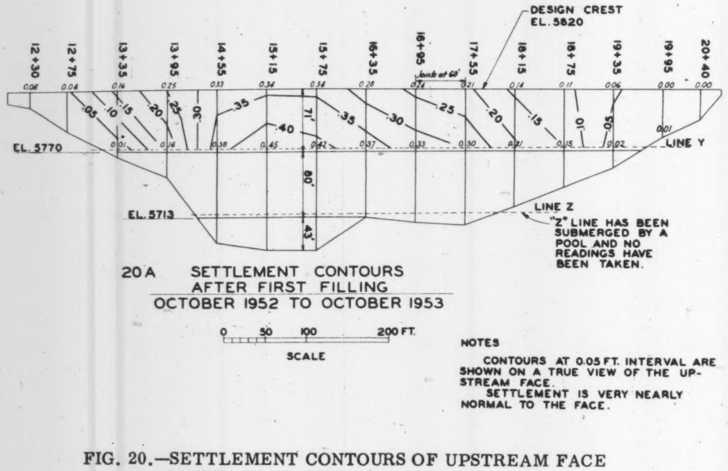
An attempt has been made to interpret and discuss the performance of the Salt Springs and Lower Bear River concrete face rockfill dams. However, the value of this paper is considered to be in the reasonably complete presentation of facts, from which the reader can draw his own conclusions.


OVERBUILD
FT. 0.2 0.4 0.6 0.8 1.0 1.1 1.1 1.0 1.0 0.9 0.9 0.7 0.4 FT. 0.1
0.10 0.12 0.31 0.43 0.52 0.49 0.52 0.52 0.41 0.35 0.30 0.22 0.16 0.06 0.08
.05 .10 .20 .30 .40 .45 .50 .50 .50 .45 .40 .35 .30 .25 .20 .15 .10 .05
0.01 0.20 0.47 0.54 0.51 0.44 0.44 0.37 0.26 0.18 0.05 0.03
LINE Y
LINE Z

20C SETTLEMENT CONTOURS AFTER FIRST 4 YEARS
SUM OF 20 A AND 20 B
OCTOBER 1952 TO OCTOBER 1956


12+30 13+35 14+55 15+75 16+95 18+15 19+35 20+40
0.06 0.08 0.15 0.18 0.19 0.15 0.18 0.14 0.17 0.14 0.16 0.11 0.10 0.06 0.00
.15 .15 .15 .15 .10 .10 .05 .10 .05
0.00 0.04 0.09 0.09 0.09 0.07 0.07 0.07 0.05 0.03 0.03 0.02
EL. 5770
LINE Y
EL. 5713
LINE Z

20B SETTLEMENT CONTOURS FOR THE 3 YEAR
AFTER THE FIRST FILLING
OCTOBER 1953 TO OCTOBER 1956


DESIGN CREST
EL. 5820
12+30 12+75 13+35 13+95 14+55 15+15 15+75 16+35 16+95 17+55 18+15 18+75 19+35 19+95 20+40
0.06 0.04 0.16 0.25 0.33 0.34 0.34 0.28 0.24 0.21 0.14 0.11 0.06 0.00 0.00
Joints at 60'
.05 .10 .15 .20 .25 .30 .35 .40 .35 .30 .25 .20 .15 .10 .05
71'
80'
43'
0.01 0.16 0.38 0.45 0.42 0.37 0.33 0.30 0.21 0.15 0.02 0.01
EL. 5770
LINE Y
EL. 5713
LINE Z
"Z" LINE HAS BEEN SUBMERGED BY A POOL AND NO READINGS HAVE BEEN TAKEN.

20 A SETTLEMENT CONTOURS
AFTER FIRST FILLING
OCTOBER 1952 TO OCTOBER 1953

0 50 100 200 FT.
SCALE

NOTES

CONTOURS AT 0.05 FT. INTERVAL ARE SHOWN ON A TRUE VIEW OF THE UPSTREAM FACE.

SETTLEMENT IS VERY NEARLY NORMAL TO THE FACE.

FIG. 20.—SETTLEMENT CONTOURS OF UPSTREAM FACE
LOWER BEAR RIVER DAM NO. 2

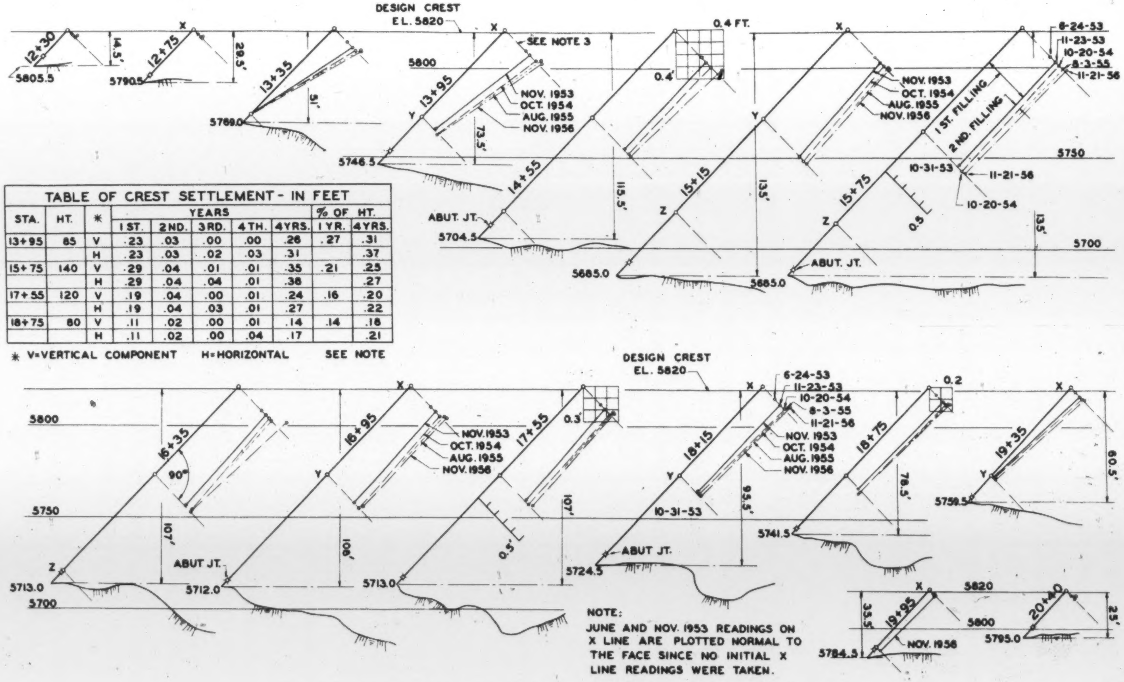

DESIGN CREST
EL. 5820
12+30
5805.5
14.5'
X
12+75
5790.5
29.5'
13+35
5769.0
51'
X
SEE NOTE 3
5800
Y
13+95
NOV. 1953
OCT. 1954
AUG. 1955
NOV. 1956
73.5'
5746.5
0.4 FT.
0.4'
14+55
ABUT. JT.
5704.5
115.5'
Z
15+15
5685.0
135'
Y
X
NOV. 1953
OCT. 1954
AUG. 1955
NOV. 1956
15+75
Z
0.5'
ABUT. JT.
5685.0
6-24-53
11-23-53
10-20-54
8-3-55
11-21-56
1 ST. FILLING
2 ND. FILLING
10-31-53
11-21-56
10-20-54
135'
5750
5700
5800
16+35
90°
Z
5713.0
107'
5750
5700
X
16+95
Y
NOV.1953
OCT. 1954
AUG. 1955
NOV. 1956
ABUT JT.
5712.0
106'
17+55
0.3'
0.5'
5713.0
107'
DESIGN CREST
EL. 5820
X
6-24-53
11-23-53
10-20-54
8-3-55
11-21-56
18+15
Y
NOV. 1953
OCT. 1954
AUG. 1955
NOV. 1956
10-31-53
ABUT JT.
5724.5
95.5'
0.2
18+75
5741.5
78.5'
X
19+35
Y
5759.5
60.5'
X
5820
19+95
33.5'
NOV. 1956
5784.5
5800
20+40
5795.0
25'

TABLE OF CREST SETTLEMENT - IN FEET

STA.	HT.	*	YEARS					% OF HT.	
			1 ST.	2ND.	3RD.	4TH.	4YRS.	1 YR.	4YRS.
13+95	85	V	.23	.03	.00	.00	.26	.27	.31
		H	.23	.03	.02	.03	.31		.37
15+75	140	V	.29	.04	.01	.01	.35	.21	.25
		H	.29	.04	.04	.01	.38		.27
17+55	120	V	.19	.04	.00	.01	.24	.16	.20
		H	.19	.04	.03	.01	.27		.22
18+75	80	V	.11	.02	.00	.01	.14	.14	.18
		H	.11	.02	.00	.04	.17		.21

* V=VERTICAL COMPONENT H=HORIZONTAL SEE NOTE

NOTE:
JUNE AND NOV. 1953 READINGS ON X LINE ARE PLOTTED NORMAL TO THE FACE SINCE NO INITIAL X LINE READINGS WERE TAKEN.

FIG. 21.—SECTIONS OF DAM AND TABLE SHOWING SETTLEMENT - LOWER BEAR RIVER DAM NO. 2

The settlement data for these concrete face rockfills is of value to engineers engaged in design of core rockfill dams as well as dams having impervious face membranes. The movements essentially normal to the face should be similar to those for a sloping core rockfill. It is noted that the measured lateral movements of the concrete face are less than those of the underlying fill due to the resistance of the slabs and joints to movement.

The lessons learned by a thorough examination of design, construction and performance of the Salt Springs and Lower Bear River Dams have made it possible to effect substantial savings in the cost of the Wishon and Courtright Dams,(29) without affecting the unquestionable safety of the dams. The design and construction of these dams is being covered in a separate paper in this Symposium.

ORGANIZATION

The design of the Pacific Gas and Electric Company's Salt Springs Dam in 1928 was under the direction of A. H. Markwart, then Chief Engineer, I. C. Steele, then Chief Civil Engineer, and Walter Dreyer, then Assistant Chief Civil Engineer. The design of the Lower Bear River Dams in 1950 was under the direction of I. C. Steele, then Vice President and Chief Engineer, and Walter Dreyer, then Chief Civil Engineer. Associated with the engineering on both dams were T. J. Corwin and G. C. Green, and on Lower Bear River Dams, J. B. Cooke.

Construction of Salt Springs was by Company forces under the direction of O. W. Peterson. Construction of the Lower Bear River Dams was by Utah Construction Company, and construction supervision by P. G. and E. Co. was directed by A. J. Swank, Vice-President in charge of General Construction and H. W. Haberkorn, Manager of Hydroelectric Construction.

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DISCUSSION

CARLOS TERCERO.[4]—Salt Springs and Lower Bear River Dams provide invaluable experience for the designers of rockfill dams, and the knowledge of the designs and the construction methods employed as well as the knowledge of the settlements and leakages recorded, as presented by Mr. I. C. Steele and Mr. J. B. Cooke, will contribute to the improvement of future dams of this type.

In Mexico, the Federal Electricity Commission built, between 1954 and 1956, the 180-ft high Pinzanes rock fill dam, for its Tingambato hydroelectric project, its main object being to provide a reservoir for the hourly regulation of the flow used for the three Francis type turbines of 72,000 HP each under an average head of 1,620 ft. Due to this high head on the power plant the water has also a high value and minimum leakage is desired.

The Pinzanes dam site is located in an active volcanic zone composed of granite type rock of greenish color and not very sound, except in a few places that were used as a quarry from which rock was blasted at three or four different levels in order to choose the best kind of rock to be sent by trucks to the dam site, and dumped from a minimum height of 50 feet. The rock sent from the quarry was not larger than 2 tons and the average size was around 0.25 tons. Much of the rock was broken by impact in the 50 ft dumping from the trucks, and the fines as well as the smaller pieces of rock were carried into the voids of the larger pieces by the water used liberally and at high velocity through nozzles fed by electric pumps. The water jets were directed to the falling rocks and later to the pieces already in place, in order to keep their surfaces free from dust or from chips and to provide a suitable contact for the next loads of rocks. The water was used at an average of 7 to 10 cubic feet of water for one cubic foot of rock.

Fig. 22 shows the plan, developed view of the face, and section of the dam. In plan the dam is curved with an 870 foot radius to the centerline of the 16.5 foot wide crest. The upstream slope is 1.2:1 and downstream slope 1.3:1. The thin concrete face is on a layer of hand placed rock of uniform 6.5 foot thickness. The slabs are reinforced with a double mesh of one-inch round steel at 14 inches centers in the lower slabs and one layer at same spacing in the slabs near the top of the dam. The joints between two slabs are of the flexible and impervious type used in the Salt Springs Dam, made with a copper sheet embedded on each slab and U shaped in the center, keeping a slot of one

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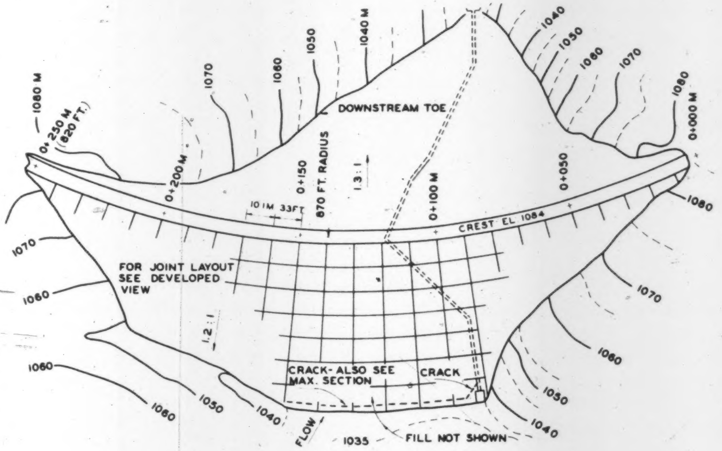
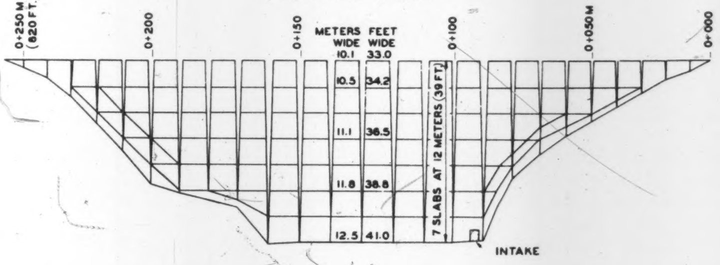
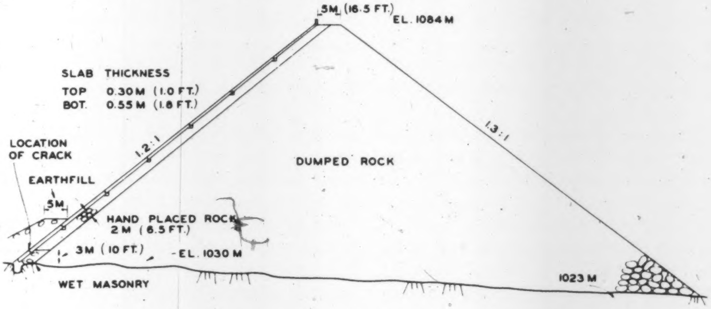
[4] Engr. of Comision Federal de Electricidad, Mexico.


1080 M
0+250 M (820 FT.)
1070
1060
1050
1040 M
1040
1050
1060
1070
1080
0+000 M
DOWNSTREAM TOE
0+200 M
0+150
870 FT. RADIUS
1.3:1
0+100 M
0+050
10.1 M 33 FT
CREST EL 1084
1080
1070
FOR JOINT LAYOUT SEE DEVELOPED VIEW
1060
1.2:1
1070
1060
1060
1050
1040
CRACK- ALSO SEE MAX. SECTION
CRACK
1060
1050
1040
FLOW
1035
FILL NOT SHOWN
1050
1040

GENERAL PLAN


0+250 M (820 FT.)
0+200
0+150
0+100
0+050 M
0+000
METERS WIDE
FEET WIDE
10.1 33.0
10.5 34.2
11.1 36.5
11.8 38.8
12.5 41.0
7 SLABS AT 12 METERS (39 FT.)
INTAKE

DEVELOPED VIEW OF UPSTREAM FACE


5M (16.5 FT.)
EL. 1084 M
SLAB THICKNESS
TOP 0.30 M (1.0 FT.)
BOT. 0.55 M (1.8 FT.)
LOCATION OF CRACK
1.2:1
1.3:1
DUMPED ROCK
EARTHFILL
5M
HAND PLACED ROCK 2 M (6.5 FT.)
3M (10 FT.)
EL. 1030 M
WET MASONRY
1023 M

MAXIMUM SECTION

FIG. 22.—PLAN VIEW AND SECTION - PINZANES DAM

inch between the slabs, that finally was filled with asphalted celotex and with asphaltic compound in the upper two inches of the slot. On the slabs located near the face of the hill additional flexible joints were provided parallel to the cutoff joint.

Along the cutoff, on the sandy bed of the river, it was necessary to excavate more than was originally planned, and the original sand and gravel was replaced with masonry more resistant to settlements than the surrounding rockfill; as there was no flexible joint on the slabs at the top of this masonry; the result was a crack on the slab soon after the concrete was poured. This crack was treated very easily with asphaltic compound and finally covered with a clay fill, Fig. 22. Another crack in the concrete was located in one slab in which the reinforcing steel was wrongly and rigidly connected with the rigid wall of the unwatering conduit located at the bottom of the dam. There is no doubt that the leakage through the face of the Pinzanes Dam is through these two cracks, that is, at the two places where the slabs were not flexible enough to follow easily the settlement of the rock fill.

When the reservoir was first filled, water soon appeared at the foot of the downstream face with the same brown color of the impounded water. Two or three days later, a second spring was located near the first, but with a colorless appearance. It was very clear that the first water came from the cracks of the concrete slabs, and that the second came through the hill. The maximum initial leakage amounted to 12.5 cfs but gradually it was reduced down to 5.5 cfs from which almost 50% comes through the foundation. The reduction was obtained by pouring small amounts of coarse sand followed by fine sand, through a 6-inch diameter pipe laid on the upstream face of the dam, from the crest to the site of the cracks.

The capacity of the Pinzanes reservoir is only 4,300 acre feet so that it can readily be unwatered, and it was easy to repair the cracks when the power plant was not yet in service; but now it is not desirable to empty the reservoir down to the very bottom to make the repairs, and although the water is valuable in Pinzanes as explained before, it does not pay to stop the plant for about two weeks for the saving of 2 or 3 cfs. Nevertheless, a method used by the Pacific Gas and Electric Company to detect the position of the cracks and to fill them without emptying the reservoir will be tried in the future.

The main object of this discussion is to point out the benefits derived from the experience obtained in the Salt Springs Dam in the design and construction of the economic and very satisfactory Pinzanes Dam. The design and supervision was made by Mr. Jesus Chavez Solano, from the Comision Federal de Electricidad of Mexico, after a careful study of the Salt Springs features and experience, especially those regarding the liberal use of water during the dumping of the rock, and in the provision of flexible joints near and paralleling the abutments.

The crest of the dam was built to a crest profile based on an overbuild of 1.5 per cent of the height of the point above the profile of the cutoff wall. However, such settlement is far from being reached, and this dam can be ranked among the most successful, so far as we know, because even after the earthquake of July 28, 1957, the dam did not show damage or cracks. The intensity of the earthquake was of the fifth degree—Mercalli System, with an acceleration between 25 and 50 millimeters per second per second.

GEOFFREY DAVEY,[5] **M. ASCE.**—The details in this paper of the settlement behaviour of the Salt Springs Dam, and the comparison of this behaviour with that of the Lower Bear River Dams, provide matter of great interest to engineers in Australia where, in many cases, rockfill is the only economical form of construction available. It is surprising to see the amount of settlement which did occur in the Salt Springs Dam during and immediately after construction. It might be asked whether such settlement would have occurred had the construction been carried out in small lifts of the order of 50' instead of the high lifts which were used. It should be possible with lower lifts to obtain full settlement in each layer during placing and hence to avoid later settlement.

The use of a reinforced concrete face on a placed stone up to 15' thick has obviously given very good performance despite the movement which has occurred in the fill, but it would be interesting to know the amount of leakage which has taken place through the dam. It would appear that in a controlled river system with numerous dams on one stream, that substantial leakage would not be a serious item, whereas such leakage would be quite serious for a water storage dam in an arid area of low rainfall.

MASATOSHI KAWASE.[6]—The author's paper which presents concise information on concrete face rockfill dams is very valuable to the engineers interested in such type of dams. The writer has prepared a brief discussion on the basic paper, presenting some data on the concrete face rockfill Ishibuchi Dam in a manner that it may be conveniently compared with that presented by the authors on Salt Springs and Lower Bear River Dams. It is hoped that the discussion may add something to the general pool of information on the subject of rockfill dams.

Ishibuchi Dam of the concrete face rockfill type, located on a tributary of the Kitakami River, was completed in 1953 by the Ministry of Construction. The main reasons for adopting this type at this site were unfavorable geological conditions and difficulty of procuring cement at that time. Fortunately, a sufficient supply of rock was available from a quarry consisting of dacite, while sufficient core materials were not obtainable nearby. Therefore, the concrete face rockfill type was adopted. Physical data are shown in Table 1.

Design Features

The mean slope of the downstream face, 1.6:1 was determined in consideration of stability against earthquakes. From the economic point-of-view, it would have been better to adopt a more gentle slope on account of the finishability of the face. The slope of the upstream face which is concave towards the reservoir varies from 1.4:1 to 1.2:1 and the axis of the dam in the plan is slightly curved convexly towards the reservoir. To effectively reinforce the parts where the differential settlement of the dam is greatest, the thickness of the placed rock, 3 m (9.8 ft.) at the crest, is increased linearly to 5 m (16.4 ft.) at the portion 20 m from the crest and it is made 5 m from this portion down to the bottom. The facing slab is of reinforced concrete and its thickness varies from 40 cm (15.8 in.) at the crest to 60 cm (23.6 in.) at the base. The amount of reinforcement was about 0.5% of the slab section and two layers were adopted in consideration of resistance to temperature changes and settlement of the rockfill dam. The facing slab was generally divided into 10 meter (32.8 ft.) squares by the horizontal and vertical joints, and exceptionally 3 to 5 m (9.8 to 15.4 ft.) horizontal joints were adopted in the part adjacent to the river bed where more flexibility was needed. The idea above mentioned is similar to that of the hinge slab adopted in Lower Bear River Dam. The plan, upstream view, cross section and details are shown in Fig. 23.

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[5] Cons. Engr., Gutteridge, Haskins & Davey, Sydney, Australia.

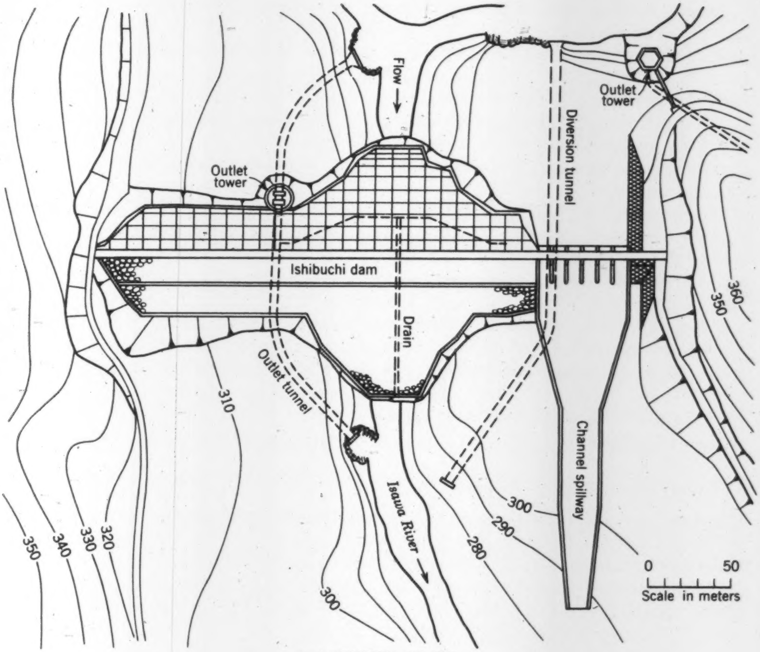
[6] Chf., River Div., Tohoku Regional Const. Bureau, Ministry of Con., Tokyo, Japan.

TABLE 5.—PHYSICAL DATA OF ISHIBUCHI DAM

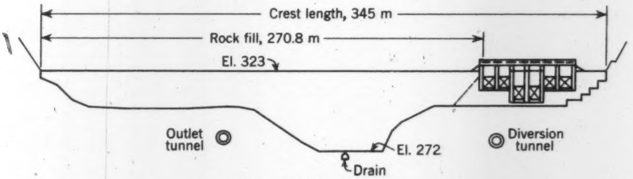
Item	Data
Year completed	1953
Effective storage m^3	11,960,000 (9,800 acre feet)
Crest elevation m	323 (1060 ft.)
Height at axis m	53 (174 ft.)
Crest length m	345 (1130 ft.)
Surface area of face m^2	11,490
Slopes (horiz. to vert.)	
Upstream	1.2 to 1.4
Downstream	1.6 (mean)
Placed rock thickness m	
Top	3 (9.8 ft.)
Bottom	5 (16.4 ft.)
Quantities m^3	
Excavation	overburden 131,500 (172,000 cu. yd.) rock 36,000
Dumped rock	361,400 (473,000 cu. yd.)
Placed rock	49,900 (65,300 cu. yd.)
Concrete (slab & keyway)	8,800 (11,500 cu. yd.)

Foundation Treatment

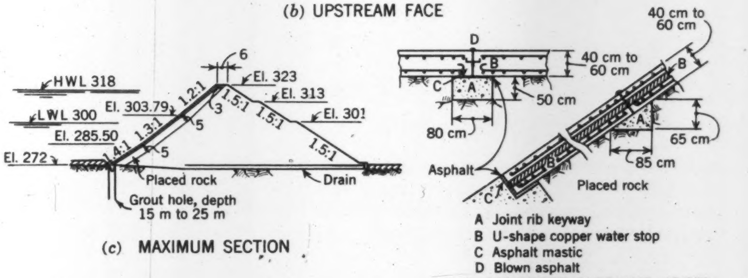
Geologically the dam site consisted mainly of liparite decomposed or jointed considerably, and the foundation rock was carefully grouted. A total of 1,587 tons of cement was injected through 9,640 meters of bored holes for curtain grouting. This shows the unfavourable geological condition of the site in comparison with that of the sites of both Salt Springs and Lower Bear River Dams.


Flow
Outlet tower
Outlet tower
Diversion tunnel
Ishibuchi dam
Drain
Outlet tunnel
Channel spillway
Isawa River
360
350
350
340
330
320
310
300
290
280
300
0
50
Scale in meters

(*a*) GENERAL PLAN


Crest length, 345 m
Rock fill, 270.8 m
El. 323
Outlet tunnel
Diversion tunnel
El. 272
Drain

(*b*) UPSTREAM FACE


HWL 318
LWL 300
El. 303.79
El. 285.50
El. 272
6
El. 323
El. 313
El. 301
1.2:1
1.3:1
1.4:1
3
5
5
1.5:1
1.5:1
1.5:1
Placed rock
Drain
Grout hole, depth 15 m to 25 m
D
B
C
A
40 cm to 60 cm
50 cm
80 cm
40 cm to 60 cm
B
A
65 cm
85 cm
Asphalt
B
C
Placed rock
A Joint rib keyway
B U-shape copper water stop
C Asphalt mastic
D Blown asphalt

(*c*) MAXIMUM SECTION

(*d*) CONCRETE FACE AND JOINT

FIG. 23.—PLAN, ELEVATION, AND MAXIMUM SECTION, ISHIBUCHI DAM

Quarrying, Dumping and Sluicing

Rocks were quarried by coyote blasting method at a quarry 1.5 kilometers upstream of the dam site and transported to the dam site on the side-dump cars which were pulled by diesel locomotives on two railroads constructed at El. 299 meters and El. 323 meters. The properties of rocks were as follows: specific gravity 2.5, absorption 4.1% and compressive strength 1,290 kg/cm^2 (18,000 psi) to 1,610 kg/cm^2 (23,000 psi). Fill-rock was dumped mainly from a bridge spanned across the river, which was constructed at the 299-meter level in the first stage and raised up to the 323-meter level in the secondary or final stage.

Uniformity of grading of fill-rock along the axis of the dam was obtained by this device of dumping. In order to expedite the settlement of rockfill in the upstream part, fill-rock was dumped on this part prior to the dumping on the downstream part. Maximum dumping height was 29 m (95 ft.) and such fines as reported on the paper of Salt Springs Dam did not accumulate on the top zone of the lift. Sluicing water was jetted in the quantity of not less than two times the volume of dumped rock and at the pressure of 7 kg/cm^2 (100 psi). However, effectiveness of sluicing by such volume of water was not sufficient. Dumping was started in May 1950 and finished in June 1953. Placing of rocks in the rubble wall section was started in July 1951 and came to an end in March 1953.

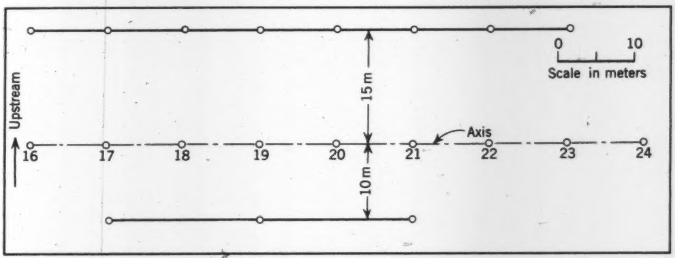
Settlement

Observations of settlement of the rockfill were conducted during the construction period at some points located on the 299 m lift and the observation data are shown in Fig. 24. Settlement of the rubble wall during the construction period was also observed at some points located on the surface of the placed rock. Location of the points are shown in Fig. 25 and maximum settlement during the construction in Table 6.

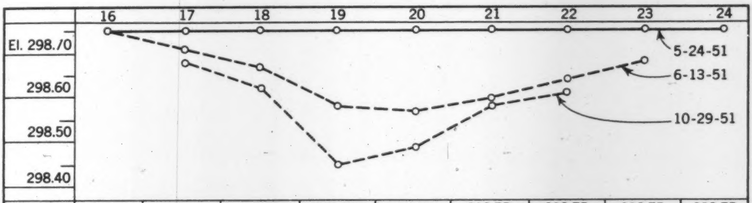
TABLE 6.—SETTLEMENT DATA OF PLACED ROCK

Location of Points	Maximum Settlement, in mm	Observation Period
Near crest-18	58 (0.19 ft.)	Oct. 6, 1952 - March 23, 1953
Near II-20	137 (0.45 ft.)	July 28, 1952 - Sept. 24, 1952
Near III-18	203 (0.67 ft.)	June 22, 1952 - Aug. 18, 1952
Near IV-74	74 (0.24 ft.)	June 22, 1952 July 21, 1952
Near V-23	23 (0.08 ft.)	June 22, 1952 - July 14, 1952

Storing of water in the reservoir began in December 1953 and the reservoir was full in April 1954 and then was maintained in the same state during the following two months. In October 1954 the water in the reservoir was drained wholly to repair the pressure tunnel for power generation and the storing was resumed in December 1954. Settlement data of concrete facing slab are shown in Fig. 26.

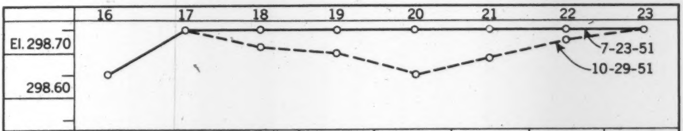


(*a*) ARRANGEMENT OF POINTS



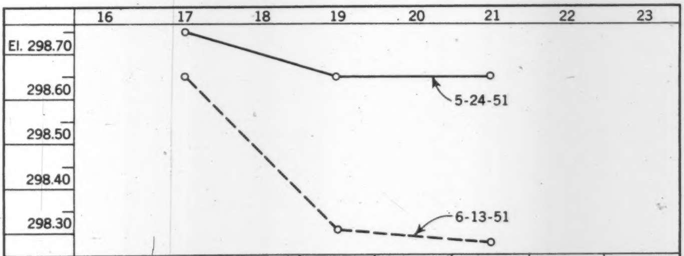
	16	17	18	19	20	21	22	23	24
5-24-51	298.75	298.75	298.75	298.75	298.75	298.75	298.75	298.75	298.75
6-13-51	298.75	298.71	298.67	298.58	298.57	298.60	298.64	298.68	
10-29-51		298.68	298.62	298.45	298.49	298.58	298.61		

(*b*) SETTLEMENT AT AXIS



	16	17	18	19	20	21	22	23
7-23-51	298.65	298.75	298.75	298.75	298.75	298.75	298.75	298.75
10-29-51	298.65	298.75	298.71	298.70	298.65	298.69	298.73	298.75

(*c*) SETTLEMENT AT LINE 15 M UPSTREAM



	16	17	18	19	20	21	22	23
5-24-51		298.75		298.65		298.65		
6-13-51		298.65		298.31		298.21		

(*d*) SETTLEMENT AT LINE 10 M DOWNSTREAM

FIG. 24.—SETTLEMENT OF EL 299 M LIFT DURING DUMPING - ISHIBUCHI DAM

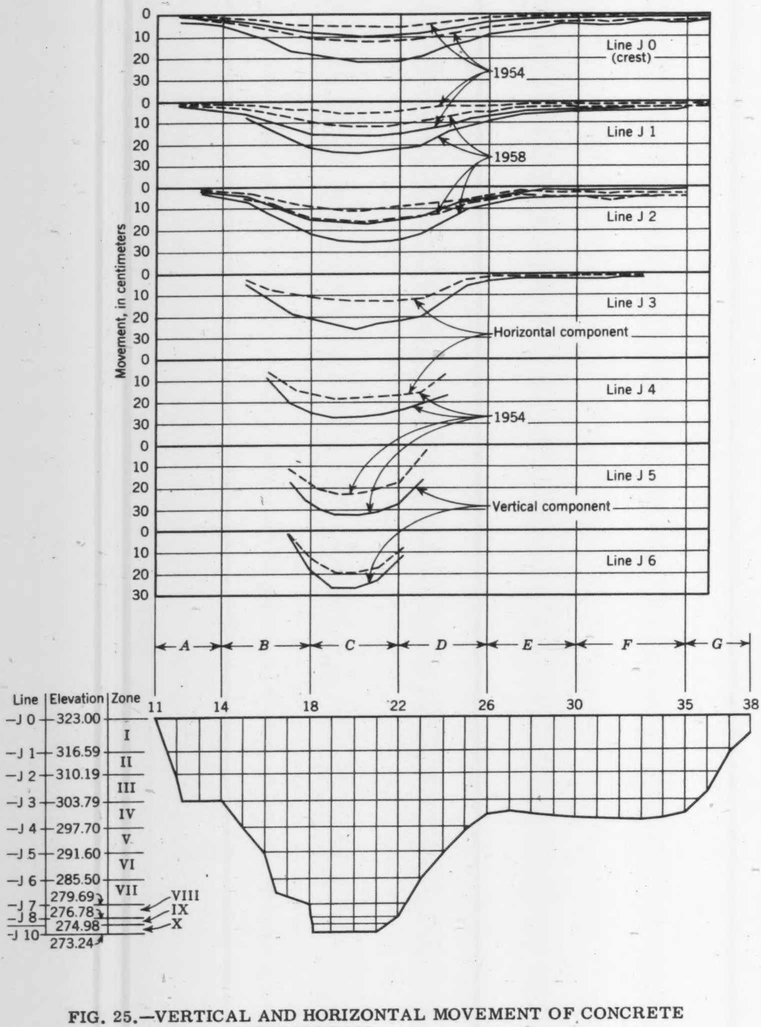


FIG. 25.—VERTICAL AND HORIZONTAL MOVEMENT OF CONCRETE FACE - ISHIBUCHI DAM

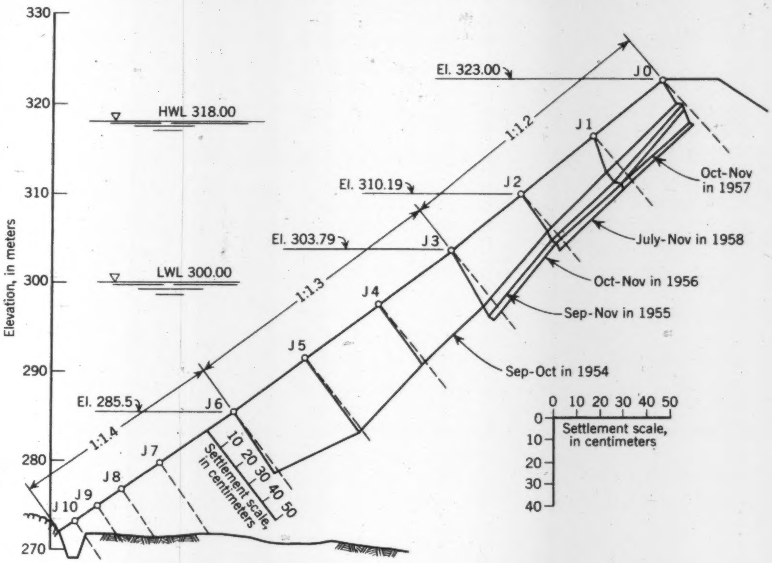


FIG. 26.—SETTLEMENT AT MAXIMUM SECTION STATION NO. 20 - ISHIBUCHI DAM

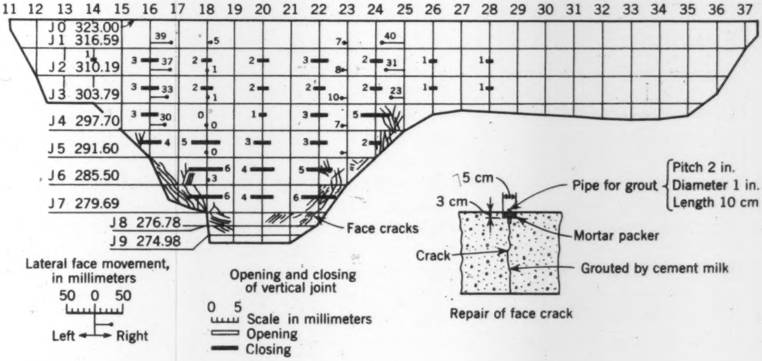

11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37
J 0 323.00
J 1 316.59
J 2 310.19
J 3 303.79
J 4 297.70
J 5 291.60
J 6 285.50
J 7 279.69
J 8 276.78
J 9 274.98
Face cracks
5 cm
3 cm
Pipe for grout
Pitch 2 in.
Diameter 1 in.
Length 10 cm
Mortar packer
Crack
Grouted by cement milk
Repair of face crack
Lateral face movement, in millimeters
50 0 50
Left
Right
Opening and closing of vertical joint
0 5
Scale in millimeters
Opening
Closing

FIG. 27.—OPENING AND CLOSING OF VERTICAL JOINT FACE CRACKS AND LATERAL MOVEMENT - ISHIBUCHI DAM

In point of relationship of settlements between the crest and the lower parts, the data show a similar tendency to those of Salt Springs and Lower Bear River Dams. It is considered that this shape of the settlement curve is due to the following reason: the settlement of the portion near the base occurs more quickly than that near the crest, where the water pressure and own weight of dam are considerably smaller than those of the other portions. Then it is considered the form of the concrete face which is concave on the upstream is favourable from the viewpoint of resistance against sliding along the surface of the placed rock.

In regard to the movement of concrete facing slab along the direction of the axis of the dam, it was found that all the vertical joints, each with an initial opening of 2 centimeters, (0.8 in.) were compressed uniformly and contracted a maximum of 0.6 centimeter (0.23 in.). Also it was found that all the horizontal joints were compressed and asphaltic materials inserted in these joints were pushed out.

Several cracks, similar to those that developed at Salt Springs, were found in the slab near the abutments in September when the reservoir was emptied (Fig. 27). Although some of them were 3 to 5 mm (0.1 to 0.2 in.) in width, the majority were hair cracks. It would have been preferable to increase the thickness of the rubble wall especially in such parts as those adjacent to the steep abutments, as well as to give the slab more flexibility such as was successfully done at Lower Bear River Dam No. 1.

Leakage

The pamphlet "Dams in Japan" describes leakage through the Ishibuchi Dam as 226 lit/min; however, this value is incorrect and should be replaced by 226 lit/sec (8.0 cfs). The leakage consists of that passing through the foundation rock and that through the rockfill dam embankment itself and it is impossible to separate one from the other. It was decreased to 204 lit/sec in 1957 and 190 lit/sec (6.7 cfs) in 1958.

TATSUO MIZUKOSHI.[7]—The authors' paper is a concise presentation of factual information on concrete face rockfill dams, especially determination of dam cross section, rock dumping, and relationship between sluicing water to dumped rock and settlement, derived from many years of successful experiences of the Pacific Gas and Electric Company by the authors.

The writer has prepared a discussion which briefly outlines some practice and construction data of the concrete face rockfill type Nozori Dam constructed by the Tokyo Electric Power Co. The facts contained herein are presented with the idea that they may supplement the valuable data presented by the authors on the Salt Springs and Lower Bear River Dams.

The Nozori Dam situated in the high mountains, about 1,500 meters (4,910 ft.) in altitude and located in the north-central part of Honshu Island. Elevation of this dam above sea level is the highest of existing dams in Japan. At this site, the working period was limited by snowfalls in winter and frequent rainfalls in summer on account of being located on the divide of the

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[7] Chf. Civ. Engr., Const. Dept., Tokyo Electric Power Company, Inc., Tokyo, Japan.

Pacific Ocean side and the Japan Sea side, and the transportation of cement and other materials was minimized from the topographical remoteness. A rockfill type was adopted from the economic viewpoint and in consideration of possible future heightening to increase the storage capacity in relation to a pumping-up scheme. And a concrete face rockfill type was selected chiefly because of the above-mentioned unfavorable climatic condition as well as unavailability of suitable impervious fill materials near the dam site. Physical data are shown in Table 7.

TABLE 7.—PHYSICAL DATA OF THE NOZORI DAM

Item	Data
Year completed	1956
Catchment area	16.6 km^2 (6.4 sq. mi.)
Effective storage	28,400,000 m^3 (23,000 acre feet)
Crest elevation	1,517 m (4,980 ft.)
Height at axis	44 m (144 ft.)
Crest Length	152.5 m (500 ft.)
Surface area of face	4,280 m^2 (46,000 sq. ft.)
Slopes (horiz. to 1 vert.)	
Upstream	1.3
Downstream	1.5 (mean)
Placed rock thickness	
Top	3.0 m (9.8 ft.)
Bottom	3.5 m (11.5 ft.)
Facing slab thickness	
Top	0.3 m (11.8 in.)
Bottom	0.6 m (23.6 in.)
Quantities	
Excavation	46,400 m^3 (60,600 cu. yd.)
Dumped rock	160,200 m (210,000 cu. yd.)
Placed rock	17,200 m (22,500 cu. yd.)
Concrete	
Facing slab	2,400 m^3 (3,140 cu. yd.)
Cut-off wall	3,900 m (5,100 cu. yd.)

Design Features

The natural slope of the dumped rock was presumed to be 1.3:1 (horizontal to 1 vertical). The 1.3:1 upstream and 1.5:1 downstream face slopes were determined in consideration of the influence of earthquakes, and for the downstream face, the 1.3:1 face slope with two berms of 4.5 meters (14.8 ft.) each in width, was actually adopted from the natural slope of the dumped rock. The thickness of the crane-placed rock, 3 meters (9.8 ft.) normal to the face for the full section, was thought to be enough, in the case of a dam of this scale, to minimize the influence of water pressure and settlement, but in the lower part the thickness varied from 3.0 to 3.5 meters. The downstream face below the lower berm was paved with the thin placed rock. A plan and a cross section of the dam are shown in Fig. 28 and 29. Actually, an overfill amounting to 2% of the dam height was adopted in consideration of the settlement of the fill-rock.

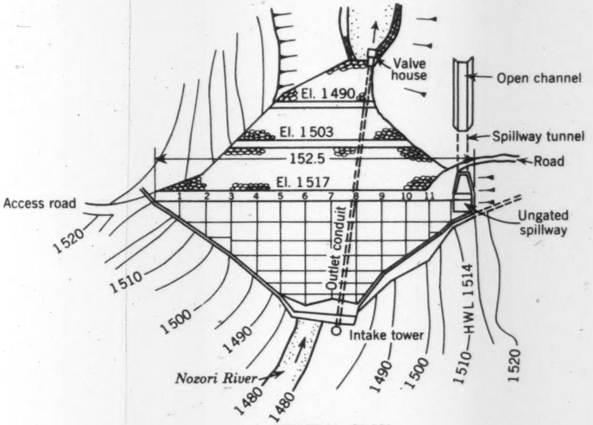
The face slab is of reinforced concrete. The slab thickness at the bottom was based on 1.5% of the water head. The amount of reinforcing bar was about 0.5% of the slab section and a single layer arrangement in the upper half and double arrangement in the lower half were adopted. The joints of the slab were arranged as shown on Fig. 28. These were divided to 12 meters (39.4 ft.) square, 12 x 8 meters and 12 x 7 meters, excepting the perimetral part adjacent to the exposed foundation rock. Details of these joints are shown on Fig. 29. Vertical joints with U-shaped copper waterstops, filled with asphalt and packed at the surface with blown asphalt containing 1.5% of asbestine fiber, are designed 25 millimeters (1 in.) open to prevent crushing of joints. Horizontal joints with flat copper plates, filled with asphalt, are also designed 10 millimeters (0.4 in.) open. These flat waterstops were considered to simplify the cross connection of horizontal and vertical joints. Joint rib keyways are coated with asphalt milk before slab concrete is placed. The U-shaped copper waterstops are used at the perimetral joints of cut-off wall and facing slab, both faces of which are also separated by asphalt fillers.

Quarrying, Dumping and Sluicing

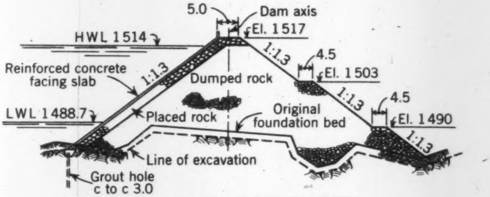
Rocks at the dam site and nearby are andesites and propylites. Quarried rocks, supplied from several quarries close to the left abutment of the dam, were obtained by the coyote hole method in 30 blasts with coyote holes totaling 1,120 meters in length. About 26 tons of Carlit (powder explosive of ammonium perchlorade) were used for blasting 215,000 cubic meters of rocks and the average explosive factor was found to be 0.27. Quarried rocks weighing 0.2 to 2 tons were used for the dumped rock. The physical properties of the rock are shown in Table 8.

The rockfill was dumped by rear-dump and side-dump cars in three stages of dumping heights, 13 to 17 meters (43 to 56 ft.). These dumping lines are at El. 1490, 1505 and 1517 meters, respectively. Rockfill was constructed in equal amounts in the two 6 month construction seasons of June - November 1954 and 1955. The maximum rate of dumping was 1500 cubic meters (1960 cu. yd.) per day.

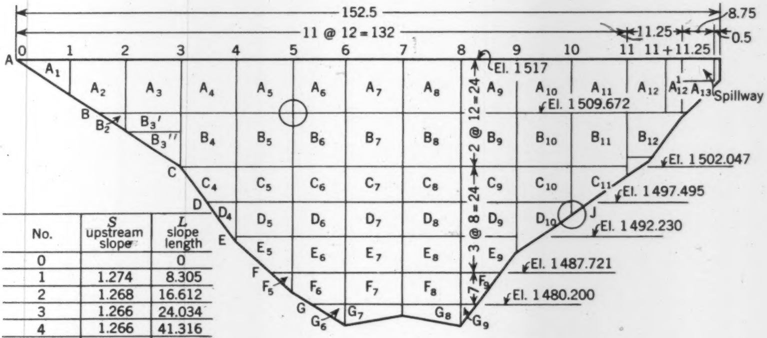
Sluicing was specified to be at not less than three times the volume of rock being dumped. Sluice water was pumped up by two 700-hp pumps from the Nozori River, supplied through two 150-millimeter dia. steel pipes to the dumping sites and poured to the dumped rock by 64-millimeter dia. rubber hoses. Every rock was washed thoroughly with this sluice water concentrated


Valve house
Open channel
El. 1490
El. 1503
Spillway tunnel
152.5
Road
El. 1517
1 2 3 4 5 6 7 8 9 10 11
Access road
Ungated spillway
1520
Outlet conduit
1510
HWL 1514
1500
1490
Intake tower
Nozori River
1480
1480
1490
1500
1510
1520

(a) GENERAL PLAN


5.0
Dam axis
El. 1517
HWL 1514
1:1.3
4.5
Reinforced concrete facing slab
1:1.3
Dumped rock
El. 1503
1:1.3
4.5
Original foundation bed
LWL 1488.7
Placed rock
El. 1490
1:1.3
Line of excavation
Grout hole c to c 3.0

(b) MAXIMUM SECTION


152.5
8.75
11 @ 12 = 132
11.25
0.5
0 1 2 3 4 5 6 7 8 9 10 11 11+11.25
A
El. 1517
A₁ A₂ A₃ A₄ A₅ A₆ A₇ A₈ A₉ A₁₀ A₁₁ A₁₂ A¹₁₂ A₁₃
Spillway
2 @ 12 = 24
El. 1509.672
B
B₂ B₃′ B₃′′ B₄ B₅ B₆ B₇ B₈ B₉ B₁₀ B₁₁ B₁₂
El. 1502.047
C
C₄ C₅ C₆ C₇ C₈ C₉ C₁₀ C₁₁
El. 1497.495
D
D₄ D₅ D₆ D₇ D₈ D₉ D₁₀
J
El. 1492.230
3 @ 8 = 24
E
E₅ E₆ E₇ E₈ E₉
El. 1487.721
F
F₅ F₆ F₇ F₈ F₉
7
El. 1480.200
G
G₆ G₇ G₈ G₉

No.	*S* upstream slope	*L* slope length
0		0
1	1.274	8.305
2	1.268	16.612
3	1.266	24.034
4	1.266	41.316
5	1.262	53.728
6	1.254	60.355
7	1.244	58.036
8	1.245	60.356
9	1.230	43.719
10	1.234	35.454
11	1.240	28.828
11+11.25	1.300	13.155

(*c*) DEVELOPED VIEW OF FACE JOINTS

FIG. 28.—PLAN, SECTION AND DEVELOPED VIEW SHOWING JOINTS OF CONCRETE FACING SLAB

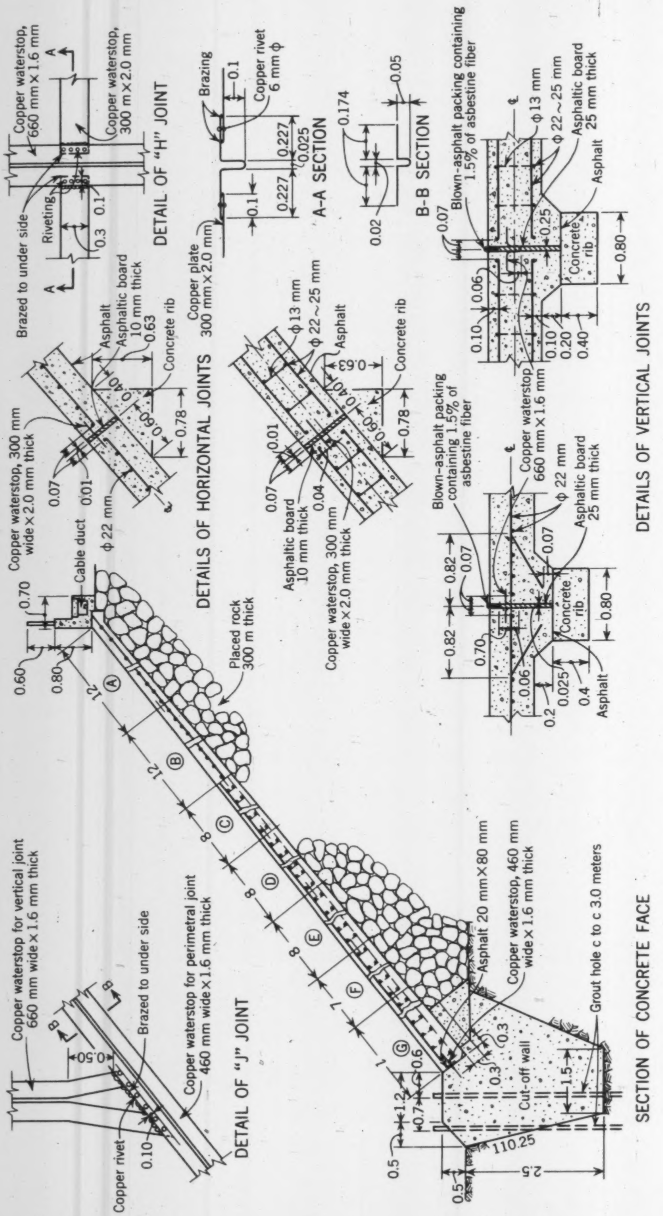

Brazed to under side
Copper waterstop, 660 mm × 1.6 mm
Copper waterstop, 300 m × 2.0 mm
Riveting
0.3
0.1
A
DETAIL OF "H" JOINT
Copper plate 300 mm × 2.0 mm
Brazing
Copper rivet 6 mm ϕ
0.1
0.227
0.025
A-A SECTION
0.174
0.05
0.02
B-B SECTION
Copper waterstop, 300 mm wide × 2.0 mm thick
Asphalt
Asphaltic board 10 mm thick
0.63
Concrete rib
0.40
0.60
0.78
0.07
0.01
ϕ 22 mm
℄
DETAILS OF HORIZONTAL JOINTS
ϕ 13 mm
ϕ 22 ~ 25 mm
0.04
Copper waterstop, 300 mm wide × 2.0 mm thick
Blown-asphalt packing containing 1.5% of asbestine fiber
Asphaltic board 25 mm thick
0.80
0.06
0.10
0.20
0.40
0.25
0.82
0.70
0.2
0.4
Copper waterstop 660 mm × 1.6 mm
DETAILS OF VERTICAL JOINTS
0.60
0.80
0.70
Cable duct
Placed rock 300 m thick
A
B
C
D
E
F
G
12
8
7
1
Asphalt 20 mm × 80 mm
Copper waterstop, 460 mm wide × 1.6 mm thick
Grout hole c to c 3.0 meters
Cut-off wall
0.3
1.5
0.6
1.2
0.7
0.5
110.25
2.5
SECTION OF CONCRETE FACE
Copper waterstop for vertical joint 660 mm wide × 1.6 mm thick
Brazed to under side
Copper waterstop for perimetral joint 460 mm wide × 1.6 mm thick
0.50
Copper rivet
0.10
B
DETAIL OF "J" JOINT

FIG. 29.—DETAILS OF CONCRETE FACING SLAB JOINTS

TABLE 8.—PHYSICAL PROPERTIES OF ROCK

Item	Date of test pieces sampled		Remarks
	Jan., 1953	Aug., 1955	
Specific gravity	2,705	2,700	
Absorption	0.311%	0.500%	
Compressive strength			
Wet condition	3,515 kg/cm^2	2,590 kg/cm^2	
Dry condition	3,189 kg/cm^2	2,446 kg/cm^2	
Weathering test	-----	0.00023%	Tested by means of Na_2SO_4 solution
Modulus of elasticity	-----	757,000 kg/cm^2	Measured Statistically

from three points as it is placed and water consumed for this purpose was about four times the volume of dumped rock. The jet pressure of the sluice water was about 12 kg/cm^2 (170 psi.).

The rocks weighing 2 to 7 tons in the placed rock section, 3 meters thick, were placed by cranes at least two months after the completion of the dumped rockfill in consideration of the initial settlement.

Leakage and Settlement

Leakage and settlement of the dam, after storing of water, from 1956 to 1958 are shown on attached drawings. In the leakage curve, Fig. 30, an unexpected sharp decline was seen at the end of October, 1956. The reason for this phenomenon appears to be that the severe cold weather brought an unexpected heavy snowfall as the result of which some of the leakage was temporarily absorbed in snow on the streambed between the dam site and the leakage weir. The maximum leakage was recorded in 1956 which was less than 23 liters per second (0.8 cfs.). The leakage tends to decrease gradually and the maximum in 1958 was only 10 liters per second (0.4 cfs).

Settlement of the concrete facing slab after completion of the dam is shown on Fig. 31 and that of the placed rock on Fig. 32a. The first-year waterload settlement is greatest around four-tenth of the height which agrees with the Salt Springs and Lower Bear River measurements. The later aging settlement is thought to be greatest at the crest, although it was impossible to observe the facing slab movement on account of snow and ice that covered the slab in Aprils of 1957 and 1958. The maximum vertical settlement at the crest occurred at the joint No. 8 and was observed as 10 centimeters (0.4 ft.), 0.23% of the dam height, about 50% of which occurred during the first year. It is believed that this small amount is due to the dumped rock being well sluiced.

Three face cracks have occurred near the perimetral joint and spillway crest in the first year of service, as shown on Fib. 32b. The vertical crack developed from slab F5 to slab D5, the maximum width of which amounted to 1.5 millimeters (0.06 in.), was about 12 meters long. The horizontal crack parallel to the perimetral joint at the bottom of the dam was about 8 meters long and its maximum opening was observed as 0.5 millimeters (0.02 in.).

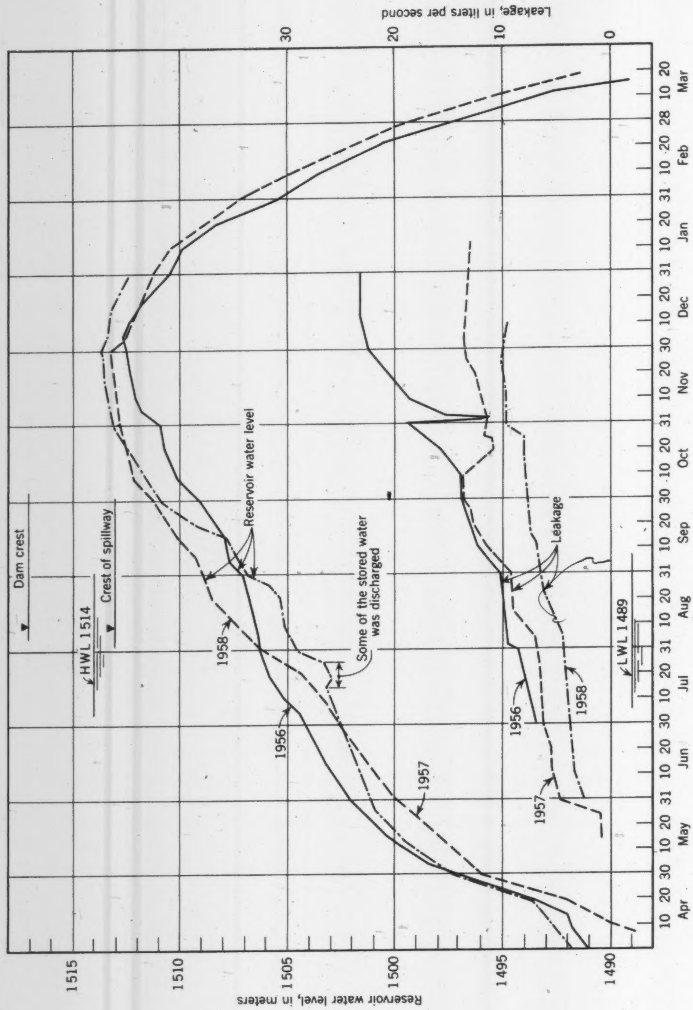

Leakage, in liters per second
30
20
10
0
Reservoir water level, in meters
1515
1510
1505
1500
1495
1490
Dam crest
HWL 1514
Crest of spillway
Reservoir water level
Some of the stored water was discharged
1958
1956
1957
Leakage
1956
1957
1958
LWL 1489
10 20 30 Apr
10 20 31 May
10 20 30 Jun
10 20 31 Jul
10 20 31 Aug
10 20 30 Sep
10 20 31 Oct
10 20 30 Nov
10 20 31 Dec
10 20 31 Jan
10 20 28 Feb
10 20 Mar

FIG. 30

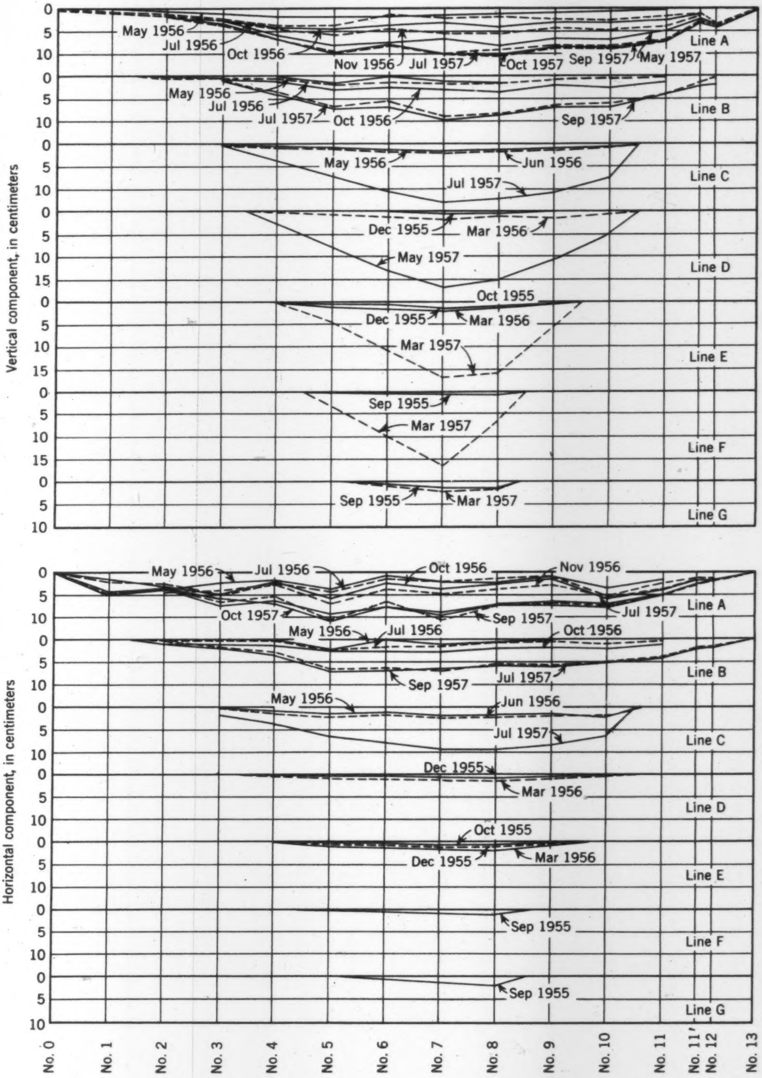
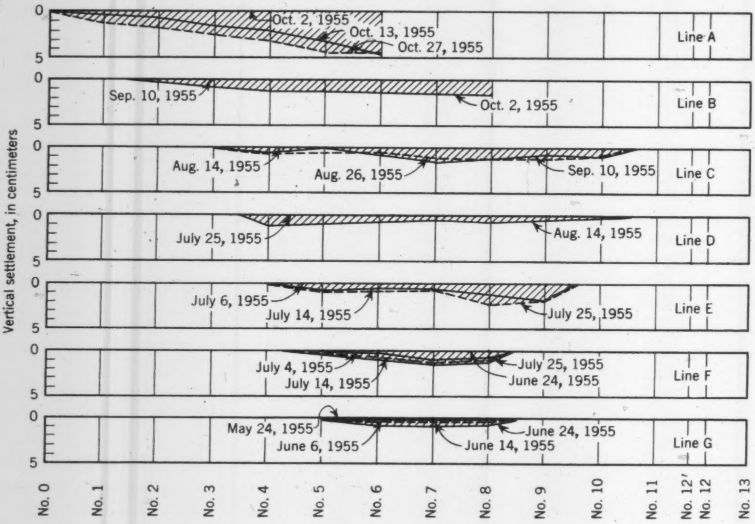


FIG. 31.—SETTLEMENT OF CONCRETE FACING SLAB



Settlement of placed rock was observed from completion of rock placing to beginning of concrete pouring of slab above

(*a*) SETTLEMENT OF PLACED ROCK DURING CONSTRUCTION

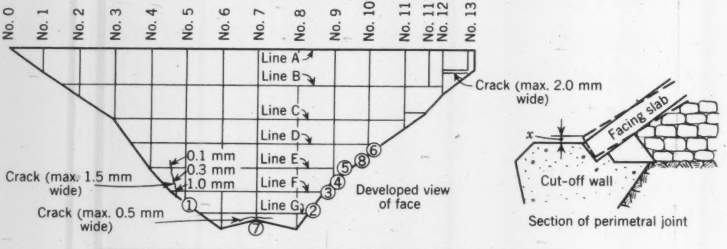

No. 0
No. 1
No. 2
No. 3
No. 4
No. 5
No. 6
No. 7
No. 8
No. 9
No. 10
No. 11
No. 11'
No. 12
No. 13
Line A
Line B
Line C
Line D
Line E
Line F
Line G
Crack (max. 2.0 mm wide)
0.1 mm
0.3 mm
1.0 mm
Crack (max. 1.5 mm wide)
Crack (max. 0.5 mm wide)
①
②
③
④
⑤
⑥
⑦
⑧
Developed view of face
x
Facing slab
Cut-off wall
Section of perimetral joint

Table of edge rise *x* of the slab in centimeter

Date observed	Joint points observed							
	1	2	3	4	5	6	7	8
Mar. 31, 1957	0	1.8	3.0	3.5	2.0	0	1.0	
Mar. 3, 1958								1.5

(*b*) FACE CRACKS AND PERIMETRAL JOINT MOVEMENT

FIG. 32.—SETTLEMENT OF PLACED ROCK, FACE CRACKS AND PERIMETRAL JOINT MOVEMENT

These two cracks occurred because of the relatively unequal movement of the fill-rock and the latter was situated above the edge of the joint rib concrete below the slab, the edge of which was found to have been raised by 1.0 centimeter (0.4 in.). The slab A13 is supported by the cutoff concrete and concrete piers on the bedrock, and the crack that occurred parallel and close to the spillway crest is unrelated to the settlement of the rockfill. The cracks have required no repair.

The performance of Nozori Dam has been very similar to that of Lower Bear River Dam No. 1, which is of similar height. It has been very successful with no maintenance being required in the first 3 years, moderate settlement and negligible leakage.

F. L. LAWTON,[8] F. ASCE.--The authors have made a particularly noteworthy contribution to the Symposium on Rockfill Dams, one which will long remain a valuable source of basic information. Being drawn from experience with 14 impervious face rockfill dams in the P.G. and E. Co. system, their location in the massive granite formations of the Sierras has added materially to the value of the data due to similarity of foundations and of rockfill.

The opening of the vertical crack near the center of the upstream face of the 140-foot high Relief Dam, with its crest curved downstream, would appear to prove the soundness of what is virtually standard practice with all types of rockfill dams, especially those with an impervious face or a thin sloping impervious core, i.e. the upstream curvature of the axis.

The authors' correlation of construction procedures with settlements and deflections in the Salt Springs Dam is invaluable for comparison with later dams, such as the Lower Bear River Dams. It drew attention to the importance of effective sluicing with plenty of high-pressure water properly directed to eliminate accumulation of fines between contact surfaces and points of the rockfill.

The vertical and horizontal rebound of points, on the maximum section of the Salt Springs Dam, even though relatively small, confirms experience with other types of rockfill dams discussed in other Symposium papers. Apparently rockfill dams act as imperfectly elastic bodies, within narrow limits. Salt Springs Dam experience over 31 years, in respect of horizontal movement in the plane of the face (lateral settlement) of the dam, affords a useful basis for assessment of possible lateral settlement in rockfill dams with thin sloping impervious cores. It demonstrates the effect of steep abutments on lateral settlement.

The authors' analysis of the inter-relationship between the several components of movement in the mass of a rockfill dam is stated as:

> "As water pressure causes significant downstream and downward differential movement, a lateral readjustment of rock points must take place. It appears that this lateral readjustment of rock contacts, along with the active pressure within the rockfill and the high percentage of voids, causes the rocks to move toward the centre of the dam. In addition to these factors the basic vertical settlement should tend to develop a component toward the center due to the sloping abutments. - -

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[8] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

It seems that the high initial lateral movements are associated with the high normal settlement in the lower and central portion of the dam."

Drawn from experience with concrete face dams, the analysis seems to be particularly applicable to rockfill dams with thin sloping impervious cores. It emphasizes the necessity for upstream curvature of a rockfill dam and the trimming of steep slopes on the abutments, as well as the elimination of sharp transitions in abutment slopes.

The average maintenance cost for Salt Springs Dam at $8,000 per year is indeed moderate. Did the $2,000,000 estimated saving, in 1931 dollars, achieved in using a rockfill rather than a concrete dam, represent construction costs only?

C. M. ROBERTS.[9]--An engineer faced with the problem of providing a rockfill dam now has available to him a wealth of information on which to base his design. Notably there are the excellently presented descriptions of the design, construction and performance of the Salt Springs Dam offered by the authors. As a result of the extensive data provided by them, the Salt Springs Dam has set a pattern for numerous subsequent concrete faced rockfill dams throughout the world.

The lessons of experience will however have best been learned by those closely in touch with the original work and their latest paper, in showing how the authors have themselves developed further designs and construction techniques to improve on the Salt Springs Dam, is therefore invaluable.

Rockfill dams have perhaps been adopted less frequently in Europe than elsewhere for the reason that concrete dams can be constructed more economically and not for any reasons of prejudice. In fact, a rockfill dam only becomes justifiable when a large proportion of the fill may be taken from other essential excavations such as a tunnel in a hydro project. This was one of the factors that made consideration of a rockfill dam possible for the Quoich reservoir in Inverness-shire, Scotland.

This dam has been fully described in a paper by the writer to the Sixth Congress on Large Dams in New York, 1958,[10] but a few remarks appear pertinent in this discussion to contrast the placing and consolidation methods used there with the dump fill method. A typical cross-section of the Quoich dam is shown in Fig. 33.

The Quoich reservoir is intended for long-term storage and it was essential that the dam should not be subject to any inherent weakness which might necessitate drawing down the reservoir for maintenance or repair work. Accordingly special measures were introduced and in particular, careful attention was given to the consolidation of the dam hearting.

As a result of discussions with the Swedish Engineers V.B.B., the writer was convinced that a rockfill dam could be constructed having virtually negligible settlement after completion of the fill. The method involved required the placing of the fill in 2 ft. thick layers and the anticipated use of tunnel spoil with consequent limited rock particle size made this a feasible proposition, permitting also the use of conventional compaction techniques. The

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[9] Partner, Sir William Halcrow & Partners, London, England.

[10] "The Garry & Moriston hydro-electric schemes," Proceedings, I.C.E. September, 1958.

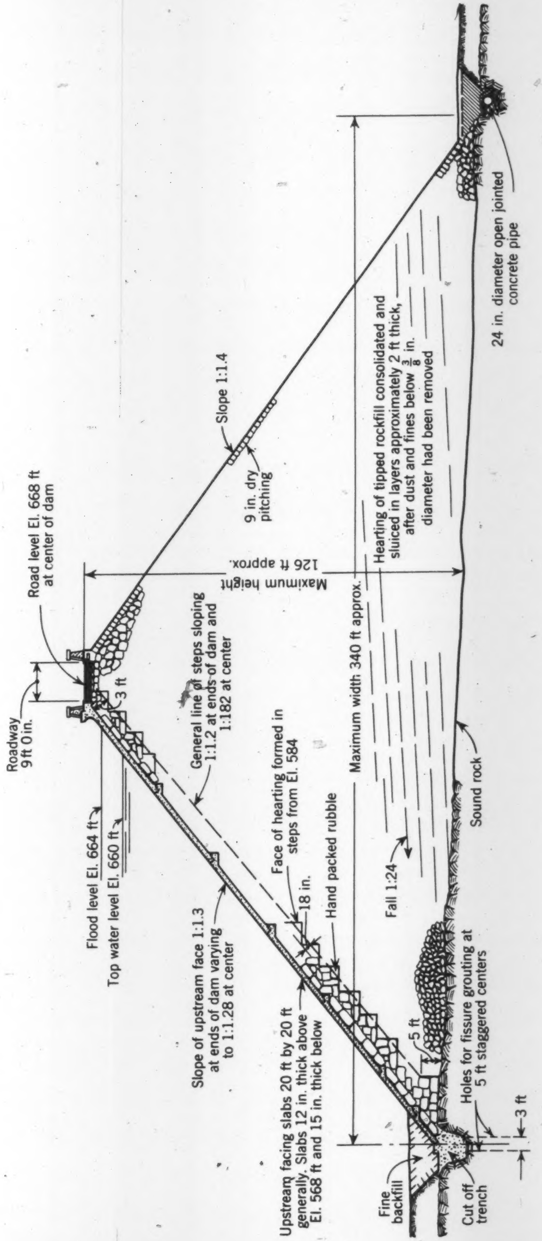
Roadway
9 ft 0 in.
Road level El. 668 ft
at center of dam
3 ft
Flood level El. 664 ft
Top water level El. 660 ft
Slope of upstream face 1:1.3
at ends of dam varying
to 1:1.28 at center
General line of steps sloping
1:1.2 at ends of dam and
1:182 at center
Upstream facing slabs 20 ft by 20 ft
generally. Slabs 12 in. thick above
El. 568 ft and 15 in. thick below
Face of hearting formed in
steps from El. 584
18 in.
Hand packed rubble
Maximum width 340 ft approx.
Maximum height
126 ft approx.
Slope 1:1.4
9 in. dry
pitching
Hearting of tipped rockfill consolidated and
sluiced in layers approximately 2 ft thick,
after dust and fines below $\frac{3}{8}$ in.
diameter had been removed
Fall 1:24
5 ft
Fine
backfill
Cut off
trench
Holes for fissure grouting at
5 ft staggered centers
3 ft
Sound rock
24 in. diameter open jointed
concrete pipe

FIG. 33.—TYPICAL CROSS SECTION OF QUOICH DAM

speed of construction was not a significant factor as the time consideration was dictated by the driving of the power tunnel.

Concern was raised by the presence of micaceous bands in the schists traversed by the tunnel giving rise to excessive fines in the tunnel spoil. A system of inspection of the tunnel face was instituted whereby a visual assessment of the proportion of mica in the rock was made prior to the firing of each round. If this was excessive the rock from that pull was considered unsound and was rejected for use in the dam. Generally however the spoil was merely passed through a simple dirt extraction plant which removed fines below 3/8'', some 16.4% of the spoil being eliminated in the process.

The clean rock was tipped onto the dam by trucks and spread by bulldozer to the required depth of 2 ft. and cross slope of 1 in 24. The passage of these vehicles over the fill gave a substantial degree of compaction before any formal consolidation measures were carried out. The first consolidation operation was rolling with a 10 ton smooth roller. In addition to pure compaction this process levelled out the surface, breaking off any upstanding points of rock, thus preparing for the second operation which was vibration with a 3-1/2 ton vibrating roller. Tests had established that vibration gave better consolidation than could be obtained with plain rolling. The vibrating roller was most effective when operating near the upper part of its frequency range i.e. 1500 cycles/min. As it approached this frequency a state of resonance occurred and the surrounding area of rockfill would shake with the vibration.

Sluicing of the surface of the fill was carried out after each stage of the compaction process to wash down into the body of the fill any fines caused by the rolling. Trial pits dug into the compacted rockfill showed that significant fines were formed but that the fill generally was compacted into a tight mass having only some 27.1% voids. The successive layer surfaces could not be distinquished in the trial pits.

The placing of the fill in layers permitted close control of the upstream and downstream slopes of the fill. In fact the upstream slope at the junction between the hearting and the hand placed stone was steeper than the natural slope and had to be supported by steps of hand placed stone set as the fill proceeded.

The effectiveness of the compaction procedure may be judged from the measured settlement after completion of the dam. The measurement points are described in the paper previously referred to. The latest observations show that the maximum settlement or deflection nowhere exceeds 3/4 inch.

In view of this result it appears that certain relaxations might be considered if this type of dam were to be repeated. The removal of fines prior to placing of the fill would be a question for judgment depending upon the nature of the rock involved. The sluicing of the fill has less significance than with the dump fill method and might be reduced. The thickness of hand placed rubble could be reduced and the sizes of the individual panels forming the concrete membrane might be increased.

KARL TERZAGHI,[11] **Hon. M. ASCE.**—This paper deals with the gradual development of the methods which were used by the Pacific Gas and Electric Co., in the construction of the fourteen rockfill dams owned by this Company. It also contains a summary of the results of the observations which were made on these dams.

Without detailed and reliable information concerning the performance of the existing dams, during construction and the subsequent service period, the construction of similar dams with an unprecedented height involves much more than a "calculated risk," and may lead to failure. Therefore by making meticulous observations on dams which were built under close supervision and placing the results at the disposal of their readers, the authors have rendered an invaluable service to the profession.

The following presentation of the paper deals with what the writer considers controversial aspects of some of the procedures which entered into the construction of the dams described by the authors. Foremost among them is the sluicing of dumped rockfills and the deposition of the rockfill in very high lifts.

Effects of Sluicing.—It is generally believed that the water discharged by the monitors washes the smaller particles into the interstices between the large rock fragments. Since the writer is unable to understand the mechanics of this process he missed no opportunity to watch the dumping and sluicing operations on rockfill dams. He arrived at the conclusions illustrated by Fig. 34, representing a vertical section through a high lift. When a new batch of rock is dumped and moves down the slope, segregation takes place whereby the average particle size and the degree of uniformity increase with increasing distance from the upper edge of the slope. Most of the "fines" come to rest on the upper part, zones a and b in Fig. 34. The mechanical action of the jets discharged by the monitors is limited to the uppermost portion of the slope extending to a distance of less than 30 ft down the slope. (Zone a, Fig. 34). Within this distance the impact of the jet moves smaller particles into the interstices between the larger ones, but the effectiveness of this action is limited to a very thin layer. This conclusion was confirmed by the following field test which was performed in the presence of the writer.

The test involved an attempt to plug the voids in a rockfill consisting of stones up to $2\frac{1}{2}$ ft in size, by sluicing fine material into the voids. For that purpose a mixture of sand and gravel was dumped onto the rockfill and then acted upon by the direct impact of a water jet, discharged at close range. In a few seconds the openings in the first row of rocks were obstructed by the largest particles of the filler and no significant quantity of filling material could be introduced in the space behind the first row. In a second test, uniform sand was washed into the rockfill but none of it went beyond the second row, because most of the kinetic energy of the jet was already destroyed while the jet was passing the first row.

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[11] Prof. Civ. Engrg., Div. of Engrg. Sciences, Harvard Univ., Cambridge, Mass.

Below zone a, Fig. 34, the water discharged by the monitors flows by gravity through the voids between the rocks, more or less vertically downward, and is unable to perform more of a mechanical action than a heavy rainstorm descending onto the fill. Hence, as a result of dumping and sluicing, the grain-size characteristics of the dike-shaped lift change from relatively fine grained and well graded material at the top to coarser and less well graded material near the bottom of the lift. Yet even within the top layer the action of the monitor is far from producing a radical modification of the texture of the fill, comparable to the effects of compaction.

In the writer's opinion the most vital benefit of the sluicing operations resides in the saturation of the desiccated outer portions of the rock fragments, where the corner breakage occurs. Saturation reduces the compressive strength of rock to its minimum value. As a consequence it increases the amount of settlement which occurs prior to the construction of the concrete facing and reduces the settlement which takes place afterwards.

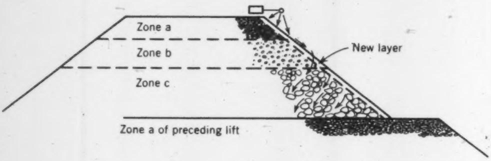

FIG. 34

The weakening influence of saturation on the compressive strength of rock has been known for many years. The ratio q'_d/q_d between the strength q'_d of the saturated rock and q_d of the dry rock is called the coefficient of softening, N_s. The following data were published more than forty years ago[12]:

Sound granite, Austria	q_d = 15,000 psi to 27,300 psi, average, 21,000 psi
	N_s = 0.74 to 0.98; average, 0.88
Granites from Sweden and Germany	q_d = 16,500 psi to 50,200 psi, average, 35,000 psi
	N_s = 0.82 to 0.99; average, 0.94
Crystalline Limestone	q_d = 8,400 psi to 20,200 psi, average, 14,000 psi
	N_s = 0.84 to 0.96; average, 0.90

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[12] "Handbuch der bautechnischen Gesteinsprüfung," by J. Hirschwald, Gebr. Borntrager, Berlin (1912), pp. 192-196.

D. McHenry of the USSR[13] obtained the following results from tests on a dense quartz-mica schist from Tennessee:

$$q_d = 13{,}590 \text{ psi}, \; N_s = 0.47$$

The preceding figures refer to perfectly sound rock. If the rock is slightly weathered the N_s - values are very much smaller. The effects thereof were demonstrated by the performance of the Cogswell rockfill dam.[14] According to the specifications, all rock "was to be sound, hard, durable, angular quarried rock, weighing not less than 160 pounds per cu foot (pcf); to be unaffected by air and moisture and of such toughness as to withstand dumping without undue shattering or breakdown; and to have a minimum compressive strength of 5000 pounds per square in. (psi)." The rock was granitic gneiss with an average compressive strength q_d = 6,629 psi. A comparison of this value with those for the granites given in the preceding table suggests that the rock was probably not quite as sound as it looked.

On December 31, 1933, when 80% of the rockfill, with an ultimate height of 250 ft, had been placed, a major storm swept over the Pacific Ocean which by noon of January 1, 1934, had yielded 15.07 in. of rain at the dam. The rainstorm caused an immediate settlement of the crest of the fill by 4% of its height (8 ft of settlement), Subsequent watering through infiltration wells increased the settlement from 4% to 6%. The settlement was ascribed to lubrication. However it is known[15] that the wetting of unpolished rock surfaces has no effect on the coefficient of sliding friction of rock on rock. Therefore, the settlement of the crest of the Cogswell Dam can only be accounted for by a very rapid decrease of the strength of the rock, produced by the first abundant rain which descended onto the fill.

Quantity of Water Required for Saturation.—If the principal benefit derived from sluicing resides in the saturation of the rock fragments, the question arises how much water is required to effect saturation. No rational procedure can be worked out for determining this quantity. However, the following facts can be used as a guide for judgement. The Cogswell Dam represents a body of rockfill with an average height of 125 ft. The initial settlement of the fill by an amount of 4% of the height of the fill was produced by 15.07 in. = 1.25 ft of water descending onto the fill, which is equal to 1% of the volume of the rockfill. The quantity of water which was subsequently pumped into the fill and increased the settlement from 4% to 6% hardly amounted to more than 10% of the volume of the fill. The quantity of water which reduced the unconfined compressive strength of McHenry's specimens of quartz-mica schist from 13,590 psi to 6,400 psi was equal to 0.3% of the volume of the specimens. Therefore it appears that the "volume ratio of water rock of 2:1" advocated by the authors (p. 102, herein) is definitely on the conservative side. Less than one-tenth of it would probably be sufficient to produce complete saturation of the rock fragments even in the event that the fragments were placed in an entirely desiccated condition. The balance of the water performs more or less useful mechanical work in the thin top layer a of the lift, but it has no influence on the properties of the rockfill in zones b and c.

Compressibility of Rockfills.—The compression characteristics of dam construction materials such as earth or rockfill include the compressibility of the materials and the time-lag associated with the process of compression. The compressibility of different construction materials can be compared by plotting diagrams such as Fig. 35 showing the relation between the unit load q on layers of the different materials, with thickness equal to unity and the corresponding total decrease d of the thickness of the layers during placement and after construction. The papers published in the Symposium, including the authors' paper, do not contain any information concerning the compression of rockfills during their construction. Therefore the data plotted in Fig. 35 are of some interest.

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[13] "Stress Conditions for the Failure of Saturated Concrete and Rock," by K. Terzaghi, Discussion by D. McHenry, Proceedings, ASTM, Vol. 45, 1945.

[14] "Rockfill Dams: Cogswell and San Gabriel Dams," by Paul Baumann, Transactions, ASCE, Vol. 125, Part II, 1960.

[15] "Erdbaumechanik," by K. Terzaghi, Franz Deuticke, Vienna, 1925, pp. 42-48.

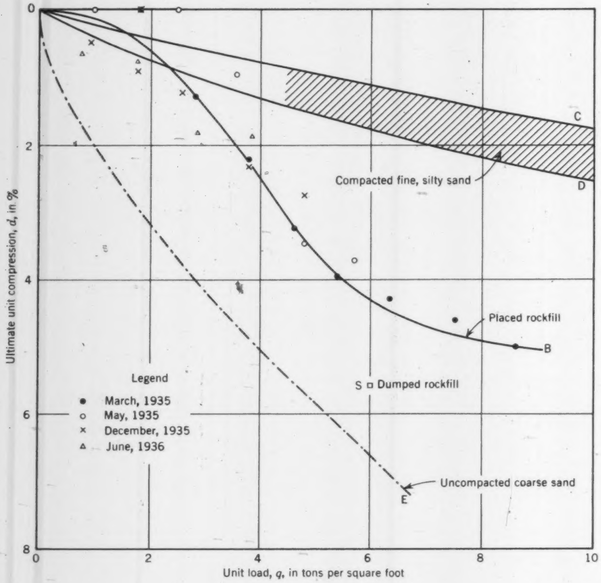

FIG. 35

The observation points and line B in Fig. 35, represent the relation between unit load and unit compression of the rockfill forming the Bou Hanifia Dam in Algeria under loads ranging between small values and a maximum of 7.5 tons per sq ft. This dam, with a height of 170 ft, was built during the years 1935 to 1937. It consists entirely of what the authors of the paper call "placed rockfill." The rock came from a quarry in sound, hard sandstone and conglomerate. The prevalent size ranged between 2 tons and 7 tons, maximum size 15 tons. The rock was placed by derricks in 15-ft lifts. The derricks were located on the upper surface of the advancing lifts and the interstices between the derrick-laid rocks were filled with small stones by native labor. The rockfill had an initial porosity of 26% to 27%. The upper part of the upstream face had a slope of 0.8 (horizontal): 1.0 (vertical), the lower part a reinforced concrete skin. The downstream slope is 1.3:1.

On the center line of the dam a cylindrical concrete tower was erected, resting on the base of the highest portion of the dam. The walls of the tower contained windows spaced about 15-ft vertically. The lowest one was located about 25-ft above the base of the tower. Through the windows the elevation of markers on large blocks of the rockfill adjacent to the tower was measured periodically while the rockfill was being constructed. During visits at the dam site the writer heard off and on a loud report, like a gun shot, coming from the direction of the rockfill. It suggested the sudden failure of large blocks in the fill.

The fill was placed at a rate of about 42 ft in height per yr. This rate is so slow that the settlement of the fill after construction was very small compared to that during construction. Therefore the observed compression of the individual layers of the rockfill is almost equal to their ultimate compression. A summary of the results of the observations on the reference points was published by J. C. Ott.[16] On the basis of these records, curve B in Fig. 2 was plotted. The results of the individual readings are represented by points.

For comparison curves C, D and E were added in Fig. 35. The space between curves C and D represents the range of the ultimate compression of the central portion of the Mammoth Pool Dam on the San Joaquin River in California. This portion consists of well-compacted weathered granite composed of chemically intact granite particles of sand and silt size. The compressibility data were derived by extrapolation from the results of observations on cross-arm installations similar to those which are used by the USBR. About two-thirds of the compression took place during construction. The diagram shows that even a placed rockfill is far more compressible than a well-compacted fill made of what can be considered a granitic silty sand. On the basis of other similar observations the writer had arrived at the conclusion that a well-compacted sand and gravel fill or compacted rockfill has far more desirable mechanical properties than a dumped, or even a "placed" rockfill. Curve E in Fig. 35 was obtained in connection with the construction of the Kenney Dam in British Columbia, by a confined compression test on crushed basalt with sand-size particles, in an uncompacted state.

By an analysis of the settlement data shown in Fig. 6 of the paper, the writer found that the ultimate total compression of the dumped rockfill forming the Salt Springs Dam under a load of 6 tons per sq ft is at least 5.5%. In Fig. 35 the corresponding point S is located between curves B and E. Since curve B was derived from observations on a "placed" rockfill, the high compressibility of the dumped rockfill, disclosed by the low position of S in Fig. 35 is by no means surprising.

Time-Lag of Settlement.—Next to importance to the compressibility is the time lag associated with the compression of rockfills. This lag requires careful consideration, because it has a decisive influence on the performance of concrete facings. The time lags associated with the compression of dumped rockfills are conspicuously disclosed by Fig. 4 of the paper, representing the three components of the movement of points of the crest of lifts. The writer first observed the time lag in the compression of cohesionless aggregates in 1920 in connection with an experimental investigation of the compressibility of cohesionless sand. Fig. 36 shows the gradual increase of the compression of a layer of loose, clean, cohesionless sand under constant load, after rapid application of the load.[17] The time lag was ascribed to the fact that the sand layer in its initial state contains many graine which came to rest, during the process of pouring the sand, in a rather unstable position. When a load is applied they rotate into more stable positions. The movement of every grain affects the conditions for the equilibrium of all the others, and such chain reactions inevitably require a considerable amount of time. Since the number of particles in relatively unstable positions decreases with time, the rate of settlement due to readjustments also decreases with time as shown in Fig. 36. The progressive settlement of rockfills is caused by similar processes.

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[16] "La Construction du Barrage de Bou Hanifia," by J. C. Ott, Algérie Bulletin de la Suisse Romande, Lausanne, February, 1946.

[17] "Erdbaumechanik," by Karl Terzaghi, Franz Deuticke, Vienna, Fig. 19, p. 103.

The settlement records shown in Fig. 4 of the paper start for every point of observation at some time after the fill was built up to the elevation of the point. The history of the compression of the fill prior to this time is inevitably unknown. However, the diagrams show that the rate of movement decreases, like that of the sand layer in Fig. 36, with time.

The movement of the points toward the adjacent advancing slope could convey the impression that "the rock going down the slope" may "pull the previously dumped fill downward" (p. 83 herein). However the drag exerted by rocks rolling down a slope is negligible compared to the force required to produce a measurable deformation of the fill. The real reason for the horizontal component of the movement of the reference points is illustrated in Fig. 37 which represents a vertical section through a layer of perfectly weightless elastic material with a sloping front. Line a b is a straight and vertical line located a short distance behind the upper edge of the slope. If the unit weight of the material is suddenly increased from zero to that of a rockfill, line a b assumes the shape a ' b whereby every point on a b moves down — and outward toward the slope.

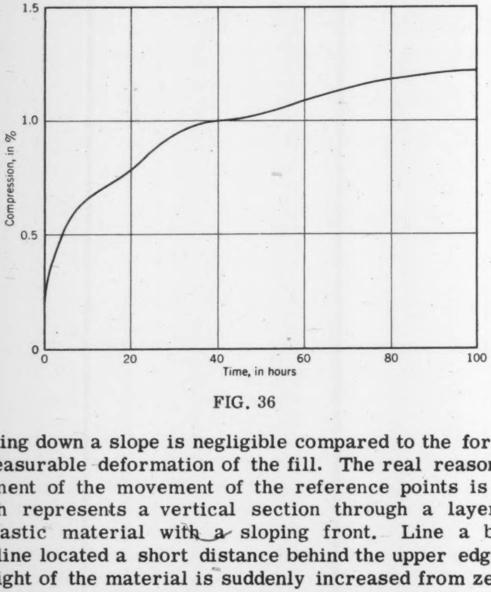

FIG. 36

In a rockfill this process of deformation goes on while the slope of the rockfill advances. The observed down-and-outward movements of points in and on the fill are the inevitable consequence of the time-lag associated with all the deformations of the fill. The authors' Fig. 4 shows that the rate of compression of the sand layer represented by Fig. 36, decreases with time.

Influence of Height of Lifts on Compression Characteristics of Dumped Fills.—On account of segregation during the process of dumping, the compression characteristics of dumped rockfills depend at least to some extent on the height of the lifts. Opinions concerning the relative merits of one-lift or multiple-lift methods of construction are still divided. In order to establish a rational basis for comparison the effects of segregation on the structure of dumped rockfills must be considered.

As shown in Fig. 34, every lift consists of three zones with more or less distinct gain-size and compression characteristics. The uppermost zone a contains the smallest particles. Its structure depends on the degree of effectiveness of the sluicing operations and may range between loose and fairly well-compacted. The thickness of this layer depends only on the technique of sluicing and, as a consequence, it is independent of the height of the lift. Zone b also contains a considerable percentage of small rock fragments, but it is located beyond the range of the mechanical action of the water jets discharged by the monitors. Therefore its structure can be fairly loose. The thickness of zone b may increase to some extent with increasing height of the lift. Zone c consists chiefly of large rock fragments which have rolled down the slope. During the process of rolling, sharp corners are likely to be broken off the fragments; this initial breakage reduces subsequent settlement due to corner breakage under static load. The corner breakage during deposition increases with decreasing compressive strength of the rock. If the rock is very hard it may be slight, and if it is weak, it may be extensive. In any event the amount of corner breakage increases with the increase of the distance through which the rock fragments have rolled down the slope. Therefore whatever favorable effects the corner breakage may have, their importance increases with increasing vertical distance from the crest.

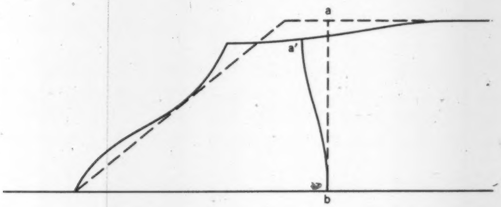

FIG. 37

On account of the consequences of segragation, the absence of significant mechanical action of sluicing below the lower boundary of the uppermost part of each lift and the increase of the beneficial effects of corner breakage in a downward direction, every lift represents a system composed of zones with different compression characteristics. In a dumped rockfill made out of a succession of lifts the compression characteristics of the fill change more or less abruptly at the level of the top of each lift, whereas in a rockfill produced by a single fill operation they change more or less gradually from the crest of the dam towards its base. Therefore the "one-lift" procedure appears to be preferable to the construction of rock fills by multiple dumped lifts. If corner breakage during deposition is a factor of consequence, on account of relatively low compressive strength of the rock, its beneficial effects increase in a one-lift rockfill in a downward direction. In a multiple-lift rockfill no such relationship exists.

In a multiple-lift dumped rockfill, the relatively fine-grained top layer of each lift is in direct contact with the largest rock fragments contained in the base layer of the superimposed lift. If the top layer contains a high percentage of fines and if in addition the compaction of this top layer is inadequate on account of deficient sluicing, the increase of the load on the large rock fragments forming the base layer of the superimposed lift causes them to sink into the fine-grained top layer of the preceding one, and thus produces excessive compression of the zone of contact between the two lifts. Such a process was probably responsible for the crushing of the concrete facing along the crest of the 165 foot lift in the Salt Springs Dam, at El. 3800. According to p. 82 of the paper the ". . . quarry method provided many large rocks but the heavily loaded 6 in. holes also produced a substantial amount of fines. . . ." "Field notes and photographs indicate that the rock coming from the quarry varied from large, clean rock just after a quarry blast to smaller rocks with many fines when cleaning up the quarry." Sluicing was also very inadequate ". . . and not available at all points of dumping" (p. 83). Hence all the prerequisites for excessive compression of the contact layer at El. 3800 were satisfied.

Influence of Height on Equilibrium Conditions of Rockfill Dams.—In some regions the rockfill type of dam is more economical than any other type, and the height of the storage dams called for by advancing civilization steadily increases. Therefore the hazards involved in constructing rockfill dams with unprecedented height most be examined.

The degree of stability of cohesionless fills is independent of height. The danger that the permeability of a very high rockfill made out of hard, sound rock such as sound granite, could be reduced by crushing to that of a fine-grained material such as sand does not exist even for a dam with a height of more than 1000 ft. Therefore the only item of concern in the construction of rockfill dams with unprecedented height is the effect of settlement on concrete facings.

For the concrete face rockfill dam maximum settlement of the facing under water load, in a direction at right angles to the upstream slope increases very roughly in simple proportion to the square of the height of the dam, because both the water pressure at the elevation of maximum settlement and the thickness of the layer subject to compression increase in simple proportion to the height. Therefore the difficulty of adapting the concrete facings to the settlement under water load increases rapidly with increasing height of the dams. However this difficulty can be eliminated by placing and compacting the rockfill in layers, whereby it acquires the favorable compression characteristics of a compacted sand-and-gravel fill. If this is done, the height of rockfill dams can be increased without serious risk far beyond the height of the highest existing dams.

Importance of Observations During and After Construction.—The large size of the rock fragments constituting rockfills precludes the possibility of predicting the performance of rockfill dams on the basis of the results of laboratory tests. Therefore the performance of such dams can be predicted with some degree of assurance only if reliable records of the performance of similar existing dams are available. Without the information contained in such records the construction of dams with a height exceeding that of their predecessors involves intolerable hazards, and as the height of the dams designed without the assistance of performance records increases it is only a question of time when the first failure will occur. On account of this fact the practical value of the observational data such as those contained in the paper can hardly be overemphasized.

The greatest uncertainties involved in the forecast of the performance of rockfill dams grow out of the absence of information concerning the amount of compression which occurs in the fill before the fill is completed. In connection with earth dams information of this kind is obtained, more and more frequently, by means of observations on crossarm installations. If a rockfill is constructed by dumping it is impracticable to measure the settlement of reference points after they are buried. However, if a rockfill is constructed by placing and compacting in layers the determination of its compression characteristics becomes practicable. This has been demonstrated by the successful settlement observations on the Bou Hanifia rockfill dam (Curve B in the Fig. 35). If the precedent established by the builders of this dam were followed by others the uncertainties which are still involved in the forecase of the performance of compacted rockfills would rapidly decrease.

Conclusions.—The controversial issue involved in the design and construction of rockfill dams are the inevitable result of the fact that the consequences of operations such as dumping and sluicing cannot be directly observed. Nobody has ever seen a cross section through a rockfill, showing the variations of grainsize and uniformity within the individual lifts, and the determination of these variations by boring, sampling and testing are utterly impracticable. Nobody has ever seen the migration of solid particles within the range of action of the jets discharged by the monitors. Also unknown are the relationships between increase in unit load and the corresponding compression of layers of rockfill during construction. Therefore the few observational data which are available leave a wide margin for interpretation. The reasoning set forth in the preceding discussion led the writer to the following conclusions:

1) The placing of rockfills by dumping is inevitably associated with segregation whereby the average size of the fragments and the degree of uniformity of the fill increase within each lift from the top toward the bottom. The degree of homogeneity of a dumped rockfill can be increased only by reducing the percentage of fines which are admitted into the fill.

2) The mechanical action of sluicing is strictly limited to the uppermost layer of the fill, where the jet strikes the slope at an angle of more than 20° or 30°, and the thickness of this layer is obviously independent of the height of the fill. Below this layer the water merely cascades from void to void without transporting a significant quantity of solids.

3) The principal function of the sluicing operations consists in the saturation of the desiccated portions of the rock fragments, which reduces their compressive strength to a minimum value. The volume of water required to achieve saturation is very much smaller than the quantity commonly used, which is equal to twice or three times the volume of the rock. The useful function of the balance of the water is strictly limited to the uppermost layer of the fill, where the jets strike the slope at an angle of more than 20° or 30°.

4) On account of segregation during dumping and the shallowness of the zone (a in Fig. 34) significantly affected by the compacting action of the water jets discharged by the monitors, every lift represents a layered system composed of horizontal layers with more or less different compression characteristics.

5) In a multiple-lift dumped rockfill the top surface of every lift represents a sharp boundary between two layers of rockfill with different compression

characteristics. Therefore the construction of rockfill dams in a single operation deserves the preference over construction in several lifts. The one-lift operation has the further advantage that the beneficial effects of corner breakage during the process of deposition increase from the uppermost portion of the rockfill in a downslope direction. The water load on the concrete facing increases in the same direction. Therefore the beneficial effects of corner breakage are greatest where they are most urgently needed.

6) Dumped rockfill is far more compressible than rockfill compacted in layers. As the height of rockfill dams is increased the detrimental effects of high compressibility increase with the square of the height. Hence for the construction of very high rockfill dams the compaction of rockfill in layers is preferable to dumping.

7) It is impracticable to determine the significant properties of rockfills by laboratory tests. These properties can only be deduced from their observed manifestations in the field. Therefore in connection with rockfill dams reliable records of the performance of existing dams, like those contained in the paper, are a prerequisite for reducing the hazards involved in the construction of dams with unprecedented height.

In conclusion, the writer wishes to express to the authors his gratitude for the wealth of valuable information which they have placed at the disposal of the engineering profession. The authors may disagree with many of the statements contained in this discussion, but if the comments succeed in demonstrating the controversial nature of many of the time-honored concepts involved in the design and construction of rockfill dams, they will have fully served their purpose.

I. C. STEELE,[18] F. ASCE and J. B. COOKE,[19] F. ASCE.—The authors thank Messrs. Tercero, Davey, Kawase, Mizukoshi, Lawton, Roberts and Terzaghi for their discussions which emphasize important points and present valuable and concise data on concrete face rockfill dams.

The 189-ft. high Pinzanes Dam, described by Mr. Tercero, is certainly very successful as well as economical with its thin placed rock and steep slopes. It is satisfying that the published experience of Salt Springs Dam was a factor in the design and that the use of sloping joints, as suggested by experience at Salt Springs Dam and as used at Lower Bear River Dam No. 1, was also successful at Pinzanes Dam. The face crack above the wet masonry at Pinzanes Dam repeats the adverse experience at Salt Springs, as mentioned in the paper, and confirms that wet masonry should not be used in lieu of placed rock above the cutoff line. That the dam was not affected by the July 28, 1957 earthquake is noteworthy. The method referred to by Mr. Tercero, used by P. G. and E. Company to locate the point or points of leakage without unwatering the reservoir, is an audio method which is based on the sound of water entering an opening in the face of the dam. Having located a leak, it can be plugged underwater by depositing a mixture of bentonite, sand and drillers cellophane; or by the work of a diver.

Mr. Geoffrey Davey raises two important questions: (1) the influence of height of lift on settlement; and (2) to what extent is the possibility and economic consequences of leakage a factor in selection of a concrete face type of rockfill dam.

Compaction of a rockfill dam can be in the form of layers of about 1.5 to 5 feet or in the form of lifts of about 25 ft to 200 ft. The thin layers are used where the rock sizes are predominately small, and in the highest core rockfill dams in zones adjacent to the transition. The placing of rockfill in compacted layers adjacent to the transition zones of a high core rockfill dam would be to reduce the size of voids as well as to minimize settlement. The selection of the adequate and economic thickness (about 1.5 to 5 ft.) of the layers depends to a large extent on the grading of rock, and on the importance of minimum settlement. Unless the rock is very small, cost would increase as the layers are made thinner. An example of the small settlement of rockfill compacted in layers is provided by the 124 ft high Quoich concrete face rockfill dam which showed no measurable settlement upon the reservoir filling,[20] and a total maximum crest settlement of only 3/4 inch after four years of service.

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[19] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

Construction of rockfill in lifts rather than compacted layers is particularly economical and is considered to be appropriate where the grading of rock includes a number of rocks in the range of 2 to 20 or more tons in weight. There are differences of opinion on the effect of height of lifts on settlement of a completed dam. The factual data on construction and settlement presented in this symposium and in the future will tend to provide a better answer to the question of the effect of height of lift on settlement of a completed dam, and also to the question of the relative settlement of a rockfill constructed in layers as against one constructed in lifts.

The concrete face rockfill dam is, of course, subject to probable leakage. The significance of this factor is believed to be minor in the economics of selecting the type of dam. Where a concrete face dam is the type of dam most economical and appropriate to the site, the capital cost saving is usually so great that leakage problems or maintenance costs are nominal factors. The first reservoir filling, or possibly the second, should see the end of any movements or stresses that would cause leakage to occur. The leakage may be located and stopped by detection and repair either underwater or in the dry. After initial repair, should it be necessary, there should be no significant future trouble.

Mr. Masatoshi Kawase and Mr. Tatsuo Mizukoshi have presented valuable data on the 174-ft. high Ishibuchi and the 144-ft. high Nozori concrete face rockfill dams. The two dams are similar to the 150-ft. high Lower Bear River Dam No. 2. The construction record shows that the rock in these three dams was dumped in high lifts and about equally well sluiced. The settlements have been similar in all respects, with maximum vertical crest settlement of about 0.25% of height of dam and on the order of 0.3 feet. The performance of all three has been particularly good from the standpoint of moderate settlement and moderate face movement; and from the standpoint of little or no leakage and maintenance.

Mr. F. L. Lawton has emphasized important points in the paper, among which is the similarity that should exist between settlement in concrete face and sloping core rockfill dams. The $2,000,000 capital cost saving of the rockfill dam over a concrete dam at Salt Springs compares with only $88,000 ($8,000 per year capitalized at 11%) that could have been spent on the concrete face dam to eliminate maintenance, or that could have been spent toward a type of dam having no maintenance. The 11% fixed charges includes about 2.5% local taxes and 2.5% federal taxes.

Mr. C. M Roberts has presented a concise discussion on the economical and most successful Quoich Dam. His inferred conclusion that the dam was perhaps too good and his stated conclusion that economies perhaps could be made in repeating such a dam are significant and important. The writers came to a similar conclusion based on the excellent performance of the Lower Bear River Dams and major economies were effected in the subsequent Wishon and Courtright Dams.[21]

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[20] "The Quoich Rockfill Dam," by C. M. Roberts, Paper R. 6, Sixth International Congress on Large Dams, October, 1958.

[21] "Rockfill Dams: Wishon and Courtright Concrete Face Dams," by J. Barry Cooke, Transactions, ASCE, Vol. 125, Part II, 1960.

for the Kenney site where all materials were specially quarried for the dam. Tunnel muck is necessarily compacted in layers but rock from a spillway quarry would generally contain enough large rocks to permit placing in lifts.

The Swedish experience, referred to by Mr. Roberts, was particularly in relation to the 50-foot high concrete face Nissastrom Dam[22] which was constructed of tunnel muck placed in layers, sluiced, rolled and vibrated. The settlement after completion of the rockfill and after filling was negligible for both Nissastrom and Quoich Dams. The two dams are not high and the compaction, as noted by Mr. Roberts, was probably more than necessary. However, the primary value of the compaction experience, as thoroughly presented elsewhere[19,21] is in its possible application to zones adjacent to the cores of core rockfill dams of major or unprecedented height. The effective use of vibration equipment on rockfills is of particular interest. The 27.1% void figure for Quoich Dam compares with 29 to 30.7% voids for the similarly compacted tunnel muck at Nissastrom Dam, and with 30 to 32% voids for a number of dumped rockfills of larger rock size.

In comparison with the 275-ft. high Dix River Dam, constructed of limestone rock, the higher 328-foot high Salt Springs Dam, constructed with granite rock, has settled less and reached a more stabilized condition. Dix River Dam vertical crest settlement has been 4.22 feet in 32 years and has been at a rate of 0.07 feet per year between the 25th and 30th year and also at a rate of 0.07 feet per year for the 30th to 32nd year. Possibly the difference in the stabilized rates of settlement for Salt Springs and Dix River Dams is due to the different characteristics of the rock in the two dams.

At the end of 7 years of service, the leakage from Lower Bear River Dam No. 1 is still less than 2.0 cfs, and the leakage from Dam No. 2 is still zero. No maintenance has been required during the 7 years. At the time of writing the paper, data on settlement for only the first 4 years of service was available. The crest settlement for the 7-year period to the end of 1959, is presented in Tables 9 and 10. No readings were taken between the 4th and 7th

Whereas Quoich Dam was made economical by the use of tunnel muck, the economics of many rockfill dams is greatly improved by laying out the spillway as a main quarry. Though such economies in utilizing essential excavations often result in the economic selection of rockfill type dams, such is not always the case as evidenced by the selection of the sloping core rockfill type

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[22] "Compaction of a Rockfill Dam," by Dr. B. Hellstrom, Paper R. 35, Fifth International Congress on Large Dams, Paris, 1955. Paper R. 35 is also available as Bulletin No. 47, Royal Institute of Technology, Stockholm, Sweden.

TABLE 9.—TABLE 3 REVISED TO EXTEND RECORD OF CREST SETTLEMENT, IN FEET, FOR LOWER BEAR RIVER DAM NO. 1

STA.	HT. AT AXIS	*	YEARS								% OF HT.	
			1 ST	2ND	3RD	4TH	5TH	6TH	7TH	7 YRS	1 YR	7 YRS
1+40	110	V	.77	.02	.02	.05	.017	.017	.017	.91	.70	.83
		H	.57	.00	.07	.00	.023	.023	.023	.71	.52	.65
3+20	220	V	1.10	.03	.05	.05	.030	.030	.030	1.32	.50	.60
		H	.74	.06	.07	-.01	.020	.020	.020	.92	.34	.42
4+40	220	V	1.03	.03	.04	.06	.030	.030	.030	1.25	.47	.57
		H	.86	.02	.07	.01	.030	.030	.030	1.05	.39	.48
5+60	150	V	.89	.02	.05	.02	.020	.020	.020	1.04	.59	.69
		H	.76	.03	.02	.01	.030	.030	.030	.91	.51	.61
6+80	85	V	.23	.01	.02	.03	.007	.007	.007	.31	.27	.36
		H	.22	.01	.03	.00	.010	.010	.010	.29	.26	.34

* V - VERTICAL COMPONENT H - HORIZONTAL

TABLE 10.—TABLE OF FIG. 21 REVISED TO EXTEND RECORD OF CREST SETTLEMENT, IN FEET, FOR LOWER BEAR RIVER DAM NO. 2 FROM 4 TO 7 YEARS

STA.	HT. AT AXIS	*	YEARS								% OF HT.	
			1 ST	2ND	3RD	4TH	5TH	6TH	7TH	7 YRS	1 YR	7 YRS
13+95	85	V	.23	.03	.00	.00	.007	.007	.007	.28	.27	.33
		H	△.23	.03	.02	.03	.007	.007	.007	.33		
15+75	140	V	.29	.04	.01	.01	.013	.013	.013	.39	.21	.28
		H	△.29	.04	.04	.01	.007	.007	.007	.40		
17+55	120	V	.19	.04	.00	.01	.007	.007	.007	.26	.16	.22
		H	△.19	.04	.03	.01	.01	.01	.01	.30		
18+75	80	V	.11	.02	.00	.01	.003	.003	.003	.15	.14	.19
		H	△.11	.02	.00	.04	.007	.007	.007	.19		

* V = VERTICAL COMPONENT H = HORIZONTAL

△ THE 1ST YEAR HORIZ. READING WAS ESTIMATED AND ASSUMED TO BE EQUAL TO THE VERT. READING SINCE NO INITIAL HORIZ. READING WAS MADE PRIOR TO FILLING RESERVOIR.

year and the movements of the 5th, 6th, and 7th years are based on assumed equal movement each year. For the 5th through the 7th years, the maximum vertical crest settlement has been at a rate of 0.03 feet per year for Dam No. 1 and 0.01 feet per year for Dam No. 2. This data gives concrete evidence of the early stabilization of settlement to a negligible amount for a rockfill dam of competent rock that is well sluiced and dumped from high lifts.

The notable paper by the late Mr. J. D. Galloway, with its discussion by a number of engineers,[23] is an important document in the history of rockfill dams. That paper in 1939 developed and stimulated interest in rockfill dams and constituted an early major contribution to the technology of rockfill dams. The papers of the ASCE Symposium on Rockfill Dams, along with the discussions by numerous engineers throughout the world, comprise a major step toward still further progress in the field of design and construction of rockfill-type dams.

Though some interpretation and discussion of the performance of the Salt Springs and Lower Bear River rockfills was included in the writers' paper, the conclusion states: "the value of this paper is considered to be in the reasonably complete presentation of facts, from which the reader can draw his own conclusions." The paper was confined to the specific dumped rockfill dams. Mr. Terzaghi's discussion develops thinking on the theory and nature of sluicing and of the mechanics of rockfills both dumped and compacted in layers.

Mr. Terzaghi suggests that "the authors may disagree with many of the statements contained in this discussion, but if the comments succeeded in demonstrating the controversial nature of many of the time honored concepts involved in the design and construction of rockfill dams, they will have fully served their purpose." His discussion is of specific value far beyond that statement. The writers do not disagree with the principles presented by Mr. Terzaghi, but rather agree and have learned something from them. However, as will be seen in the following reply to his discussion the writers give a different emphasis to the importance and/or economics of some of those principles. These differences may not in fact exist since Mr. Terzaghi's discussion was directed toward theory and toward understanding that would lead to still higher rockfill dams of unquestionable safety.

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[23] "The Design of Rockfill Dams," by J. D. Galloway, Transactions, Vol. 104, 1939. With Discussions by: Cecil E. Pearce, H. B. Muckleston, Harold K. Fox, Charles H. Paul, A. Floris, Howard F. Peckworth, Oren Reed, Walter L. Huber, Samuel B. Morris, L. F. Harza, Paul Baumann, O. W. Peterson, George W. Howson, John E. Field, John H. Wilson, Frederick H. Fowler, I. C. Steele and Walter Dreyer, F. Knapp, Ralph J. Reed, F. J. Sanger, C. S. Jarvis, L. R. East, Francisco Gomez-Perez, Miguel Jinich, and J. D. Galloway.

Effects of Sluicing.—From the standpoint of mechanical rearrangement of the particles in the dumped fill, sluicing is considered unnecessary in a rockfill of large rock with no fines and very necessary in a dumped fill containing fines and spalls. It is agreed that the mechanical effect of sluicing is confined to excavating surface areas of fines in the upper zone of the dumped slope. By washing the fines into the voids and by scattering the spalls and fines into surface pockets, the probability of obtaining structural contacts between the larger rocks is increased. The operation, even with continuous effort and supervision, is not perfect and some pockets of fines and spalls will remain as tolerable imperfections in the structural rockfill. In the lower zones of the dumped fill the segregation results in large clean rock and sluicing in those zones is not necessary. An example of a very satisfactory 140-ft high dam of clean unsluiced granite rock is Fordyce Dam.

The foregoing description of sluicing is for a rockfill containing a substantial amount of large rock. If the texture of the upper zone of the sloped surface of the dumped fill is such that the monitors cannot maintain a surface of reasonably clean rock, and if surface slides of small rock and fines occur to put such materials out of effective range of the monitor, either of two things has occurred: (1) too many loads of fines have been accepted and waste of loads of small rock is necessary; or (2) the rock is generally of such small particle size that is should have been compacted in layers. The compacted rockfill of small rock would be better than a dumped rockfill of large rock, but it would cost more. The writers agree with the statement, "yet even within the top layer the action of the monitor is far from producing a radical modification of the texture of the fill, comparable to the effects of compaction."

Mr. Terzaghi states, "In the writers' opinion the most vital benefit of the sluicing operations resides in the saturation of the desiccated outer portions of the rock fragments, where the corner breakage occurs." This is an interesting point and perhaps should be given particular attention where there may be a question of competence of the rock for a rockfill. For the fills of very large rocks of competent Sierra granites the mechanical action of sluicing has been considered the most vital and primary benefit, though it must be admitted that the relative magnitude of these factors in producing settlement during construction is not known. Even if there was no benefit from "softening" of the rock edges it is considered that mechanical sluicing is essential in a fill containing fines and spalls.

The component amounts of settlement that can be attributed to the various actions of the rain and post construction jetting of the rockfill in Cogswell Dam are, of course, not known. Mr. Terzaghi attributes a major part to the decrease in strength of the rock due to saturation. For the relatively low strength rock (6600 psi) that may well be a significant factor. For Cogswell Dam the specifications prohibited sluicing and limited lifts to 25-ft heights. Certainly the softening of the unsluiced and uncompacted pockets of fines and of the layer of fines at the top of each 25-ft lift would have contributed to the settlement. The large rock on top of each lift surface of fines would tend to bite into the softened layer, and pockets of fines which carried load in a dry condition would yield and wash away due to the saturation and water action. Additional breakage along weak cleavage planes of the granite gneiss rocks during this period of rapid settlement of rockfill may have been a contributing factor. It is agreed that lubrication should not be a factor.

Quantity of Water Required for Saturation.—"If," as Mr. Terzaghi says, "the principal benefit derived from sluicing resides in the saturation of the rock fragments, the question arises how much water is required to effect saturation." The answer appears to be, very little. A further question would be, how might the water most effectively be applied? It would seem that the application of substantial water at the point of dumping would be very effective in keeping the fill wet and humid for the full height under the point of dumping, where the loading is increasing and maximum movements in the fill would be taking place. Thus the conventional sluicing practice would serve to keep a substantial and important zone of rock in a moistened condition during construction. In addition, if it were decided that maintaining saturation during construction were of primary importance, should not the specification require spraying of the surface of the rockfill, or perhaps the loose rock in the quarry?

As noted in the paper, sluicing at a water-rock ratio of 2:1 cost 4.6 cents per cu yd of rock in the dam, and decreases or increases 0.4 cents per cu yd for changes to 1:1 or 3:1. For the small incremental cost a substantial amount of water seems appropriate. A 2:1 requirement is practicably satisfied by monitors with 2.5 to 3 in. nozzles operated at 70 to 100 psi at points of dumping. Such jets are effective in excavating and in providing a substantial volume of water where it is useful.

Compressibility of Rockfills.—Construction settlement data similar to that for the 170-ft high Bon Hanifia Dam of placed rock is available for the 124-ft high Quioch Dam of rockfill compacted in layers.[9] The Quioch dam was compacted in 2-ft layers, had very little settlement during construction and zero crest movement upon the filing of the reservoir. This is a well documented example substantiating Mr. Terzaghi's confidence in rolled rockfills.

It is the writers' opinion that a good dumped fill is as good or better than placed rock. From comparison of settlement of P. G. and E. Company dumped fill dams with French, Algerian and Russian placed-rock dams, the dumped-rockfill post-construction settlement is less. Apparently the impact contacts of the dumped rock are more competent than the gentle placement contacts of the placed rock. This has been a factor in the reduced thickness of placed rock in the most recent P. G. and E. Company dams. The 5.5% figure used in Fig. 2 for Salt Springs Dam is an approximate figure and the Bon Hanifia figures, though carefully obtained, are for one dam only. It is believed that Fig. 2 could be interpreted to indicate that the unit compressions of placed and dumped rock are comparable and are substantially more than that of material compacted in layers.

Though rockfill compacted in layers will have a lower coefficient of settlement than dumped rockfill it does not mean that dumped rockfills are not fully satisfactory and economic in most cases.

Previously, in this closure, the writers have referred to three concrete face dams of 174, 150 and 144 ft height constructed to sluiced rock dumped from high lifts that settled only 0.3 ft at the crest upon reservoir filling and have given excellent performance. There are many successful 300 to 400 ft high rockfill dams of dumped rockfill. The writers visualize the importance of rockfill compacted in layers to be (1) in utilizing rock of too small a size for a dumped rockfill and (2) in a specified structural zone or zones in dams exceeding about 400-ft height.

The writers would like to correct the statement under the heading "Dumped Rockfill: Settlement During Dumping" in accordance with Mr. Terzaghi's comments. The dumping of single loads would, of course, have little effect other than near the surface. The surface slides, referred[23] to by J. B. Cooke, provide valuable compaction service but would still affect the basic movement of the fill to only a nominal distance from the surface. Mr. Terzaghi's Fig. 37 nicely demonstrates the movement of a rockfill toward the sloped face as it settles downward. This movement is characteristic regardless of how the fill was put together. Some further supporting data is:

a. Placed rock dams of Electricite de France have bulged in the lower portion of the downstream face. This has been measured and also is visibly apparent in bulging and cracking in the previously smooth placed rock surface.

b. Measurements on the lower portion of the downstream face of Courtright Dam before filling show the outward components to be substantially greater than the downward component. Also, measurements on the lower portion of the upstream face of Wishon and Courtright Dams during construction show this bulging near the base of the fill.

Fig. 4 of the paper is useful in helping to illustrate another aspect of settlement of a dumped rockfill: that the lateral, cross-canyon, movement is greater in the direction opposite to that of dumping the rockfill. This is reviewed in another discussion by J. B. Cooke.[24] At both Kenney and Paradela Dams the construction was from one abutment, and the lateral movement of all points was opposite to the direction of dumping rock into the dam. Looking at Fig. 4 of the paper it is seen that the compaction during construction is a maximum in the direction of a line down the dumped slope. The contacts of the rocks in a line down the dumped face would also tend to be the most competent contacts. However, looking at settlement points (3) and (4) it is seen that their direction of lateral movement changes when the dumping berm of the 185 ft fill was about 200 ft away. At this time the dumped slope appears to have lost its influence on settlement at points (3) and (4) and the downward settlement is accompanied by a lateral readjustment in the direction of least previous compaction, that is, in a direction normal to the dumped slope. The action would be similar for the lateral movement due to readjustments in the fill while the water load is directly causing vertical and downstream movement therein.

Influence of Height of Lifts on Compression Characteristics of Dumped Fills.—The writers agree in principle with Mr. Terzaghi's description and conclusions regarding heights of lift and segregation. Experience with rockfills quarried from massive granite and containing a number of 15 to 25 ton rocks has been that segregation is not pronounced. Though the fines stay at the top, and the toe consists of large clean rock, many large rocks come to rest on the dumped slope as noted in the paper, "Fig. 12 shows the character of the rockfill. Note that though large rocks segregate near the toe, there are many that remain in the upper portion of the lift."

The writers tend to prefer reasonably high lifts for structural and economic reasons but accept lower lifts where construction reasons make them economic. In very high lifts of 200 to 300 ft the lateral movements due to unequal construction compaction, as discussed above in connection with Fig. 4 of the paper, require consideration. As noted, previously in this closure, "The factual data on construction and settlement presented in this symposium and in the future will tend to provide a better answer to the question of the effect of height of lift on settlement of a completed dam, and also to the question of the relative settlement of a rockfill constructed in layers as against one constructed in lifts."

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[23] Discussion by J. B. Cooke of "Rockfill Dams: Paradela Concrete Face Dam," by Luis Henrique Gomes Fernandes, Edgard de Oliveira, and Nuno de Vasconcelos Porto, Transactions, ASCE, Vol. 125, Part II, 1960.

[24] Discussion by J. B. Cooke of "Rockfill Dams: Kenney and Cheakamus Dams," by William G. Huber, Transactions, ASCE, Vol. 125, Part II, 1960.

The lift surface at Salt Springs were not well cleaned up and many fines had accumulated due to the high lifts. The diagnosis of the causes of trouble at El. 3800 by Mr. Terzaghi is the same as that of the writer. The conditions of many low lifts, no cleanup, and no sluicing at Cogswell Dam are all factors that probably made this a contributing factor to the severe settlement of Cogswell Dam, as caused by heavy rainfall. This weakness is considered to be reduced to a point of no significance by application of the following extract from the specification for Wishon Dam: "In order to provide good rock to rock contact between lifts and assure a minimum amount of settlement, Contractor shall scarify, sluice, and otherwise prepare tops of completed lifts, before the succeeding lift is constructed, in order to obtain a rough exposed surface that is free of fines and that contains only reasonably clean rocks." However, lift surfaces are specified to be at a horizontal joint which is made "soft" to permit some closing. Cooke has described[25] the lift surface treatment at Wishon Dam.

Influence of Height on Equilibrium Conditions of Rockfill Dams.—It is considered that rockfill dams can be constructed essentially to as great heights as other types of dam, with development in reasonable steps and with the use of full knowledge of all predecessor dams. The design problem is agreed to be settlement.

The maximum settlement of the concrete face under water load would be exactly in proportion to the square of the height if the modulus or compressibility of the rockfill were the same for the different magnitudes of loading on the rockfills of different compaction and of different existing stresses. Assuming identical rock, the change in height would change the initial compaction and stresses upon which the water load is superimposed. Certainly the modulus of the rockfill will vary in some manner with the different water loads and different existing stress conditions in the rockfill. The modulus would further be different and would vary differently for different rocks and different grading of rocks. Based on records of dumped and sluiced rockfills of granitic rock, Fig. 38 indicates that the maximum settlement at right angles to the upstream slope for the dams of 220 to 360-ft height is proportional to height to the 2.1 power. "Assuming" that the modulus of rockfill as stressed in a 450-ft high dam is the same as that for the lower dams, a 450-ft-high dam of well-sluiced granite would have an estimated settlement of about 9.2 ft., as determined in Fig. 38. The lower 135-ft dam is disregarded since the modulus appears to be high for the relatively small water load. The very small movement is characteristic of dams under about 150 ft.

Previously in this closure comments regarding the desirable use of compacted rock in high rockfill dams were, "The thin layers are used where the rock sizes are predominantly small, and in the highest core-rockfill dams in zones adjacent to the transition - - - the primary value of the compaction experience (at Quoich Dam) is in its possible application to zones adjacent to the cores of core-rockfill dams of major or unprecedented height - - the effective use of vibration equipment on rockfills is of particular interest." The zones adjacent to the transition were visualized to be (1) a minimum structural section of compacted rock downstream from the core to take load from the core with minimum yielding and to have minimum settlement, and (2) a smaller upstream zone of minimum settlement and of rockfill with small voids. Outside of the required compacted zones dumped rockfill of large rock should be

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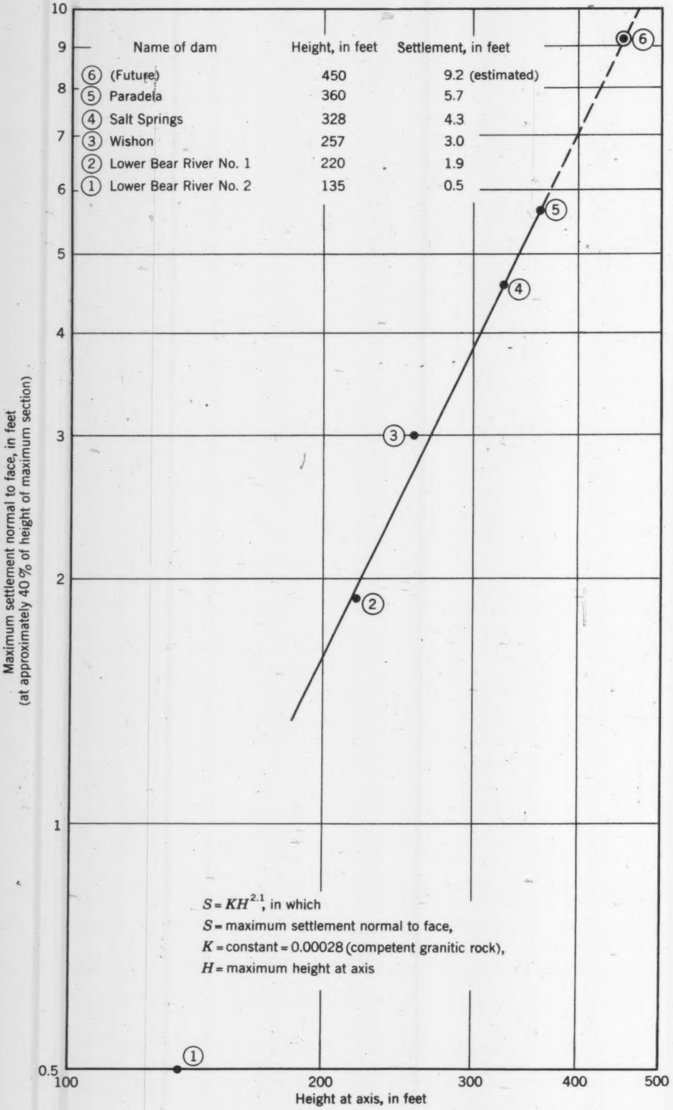
[25] "Rockfill Dams: Wishon and Courtright Concrete Face Dams," by J. Barry Cooke, Transactions, ASCE, Vol. 125, Part II, 1960, Fig. 7.



Name of dam	Height, in feet	Settlement, in feet
⑥ (Future)	450	9.2 (estimated)
⑤ Paradela	360	5.7
④ Salt Springs	328	4.3
③ Wishon	257	3.0
② Lower Bear River No. 1	220	1.9
① Lower Bear River No. 2	135	0.5

$S = KH^{2.1}$, in which

S = maximum settlement normal to face,

K = constant = 0.00028 (competent granitic rock),

H = maximum height at axis

FIG. 38.—MAXIMUM SETTLEMENT NORMAL TO THE FACE OF CONCRETE FACE ROCKFILL DAMS AFTER SECOND RESERVOIR FILLING

satisfactory and economic. In the downstream portion of the rockfill shell dumped fill of large rock would be preferred to compacted rock.

To re-phrase and amplify the previous comments, and integrate the various comments of Mr. Terzaghi, the following might be said: rockfill compacted in layers is considered appropriate in a substantial zone or zones adjacent to the filters of a core-rockfill dam of unprecedented height in order to: (1) have known composition and properties in the most important structural portion of the dam, (2) take the load from the core with minimum yielding, (3) have minimum settlement, and (4) provide a zone of known and small size voids adjacent to the transitions. The prudent thickness of core for core-rockfills of unprecedented height is important but beyond the scope of the paper and the discussion.

Importance of Observations During and After Construction.—Continued effort to investigate and understand dumped and compacted rockfills is necessary to assure the safe and economic construction of rockfill dams of maximum and of ever increasing height. As carefully and strongly pointed out by Mr. Terzaghi, information from performance records during and after construction is of critical importance to the designer of dams of maximum or unprecedented height.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3066

ROCKFILL DAMS:
NANTAHALA SLOPING CORE DAM

By James P. Growdon,[1] F. ASCE

With Discussion by Messrs. F. L. Lawton; Waldo G. Bowman; and James P. Growdon

SYNOPSIS

The design of a 250-foot high sloping core rockfill dam, part of a hydroelectric project, was determined to facilitate construction with materials readily available at the site. The paper describes the site and discusses construction procedures and performance.

INTRODUCTION

The Nantahala Hydroelectric Development is owned and operated by the Nantahala Power and Light Company, a subsidiary of the Aluminum Company of America. It is situated on the Nantahala River, in southwestern North Carolina, where steep slopes permit a high head development in a relatively short reach of river. The development consists of an earth-faced rock-fill dam, 250 feet high, creating a reservoir with 126,000 acre feet of useful storage, a spillway, a diversion tunnel, a conduit from the dam to the powerhouse, consisting of tunnels and a penstock, and a single 60,000 hp. hydroelectric generating unit operating under a maximum static head of 1,008 feet. The construction of the project was started in July 1940 and completed in July 1942. It has been in successful operation since that date.

The dam site is at the upper end of the Nantahala gorge, where the river flows between a rocky promontory on the left bank and a 40-degree solid rock slope on the right bank. (Figures 1 and 2.) The rock at the dam site is a metamorphic sandstone, called arkose. While the bed rock has been folded it occurs at the dam site in massive strata with few cracks or seams. It is exceptionally water-tight. It is an excellent site for any type of dam, except an

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Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1742. Positions and titles given are those in effect when the paper was approved for publication in Transactions.

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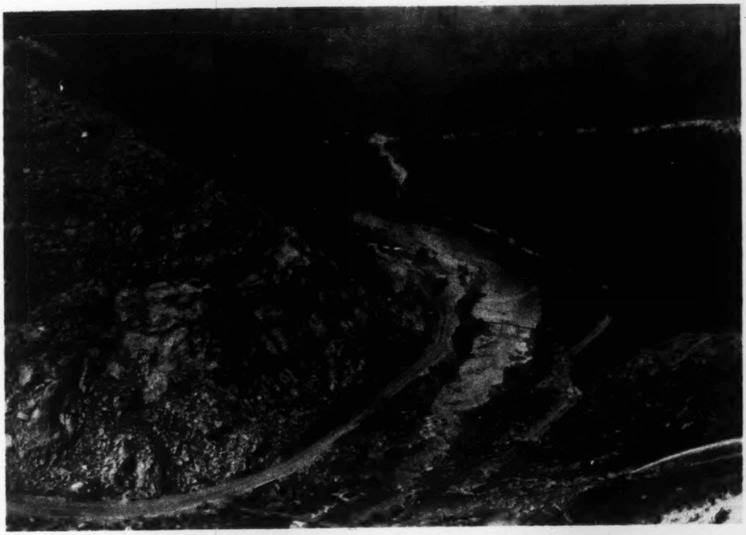

FIG. 1

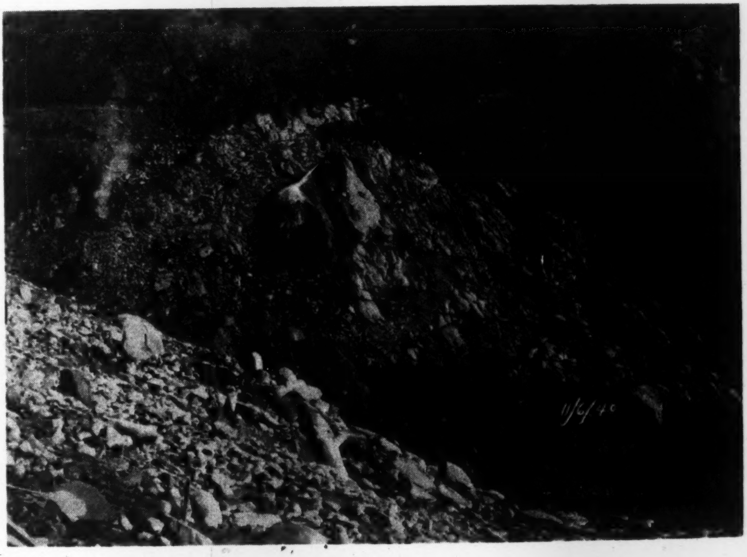

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FIG. 2

earth dam for which suitable material is not available. The dam site was acquired by the parent company in 1911. Since that date many different types of dams, including concrete gravity, combination gravity and arch, pure arch, multiple arch, Ambersen, and rock-fill have been designed for this site. Every type studied could be made to fit the site reasonably well, with varying degrees of effectiveness and economy. The rock-fill dam appeared particularly advantageous. Rock-fill dams as of 1935 had some type of a more or less rigid impervious upstream face which appeared likely to crack due to continued settlement of the rock fill. A flexible impervious face capable of adjusting itself to settlement appeared desirable.

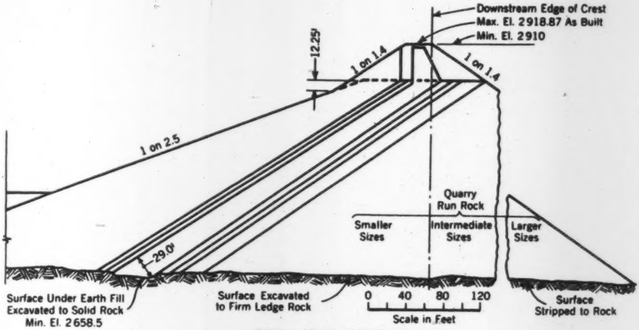
Many of the lakes in northern United States and Canada have been formed by glaciers. The glacial moraine filling up old stream beds made effective dams. Generally speaking, they consist of coarse gravel, with progressively finer material upstream and are sealed at the upper surface by glacial silt. Most of them are surprisingly water-tight and have functioned since the glaciers disappeared, - a period of several thousand years. It appeared feasible to use the same technique in a man-made structure. Further study of the problem indicated that a suitable filter would prevent the migration of an impervious earth core through the pervious rock-fill and that a reverse filter covered with rock would protect the earth core from wave action and sudden drawdown.

Design

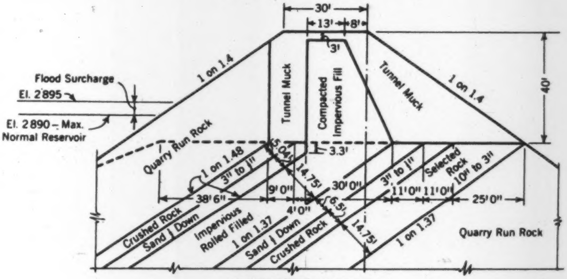
This conclusion was checked by a small model, consisting of a 12-inch steel pipe closed at the bottom with a perforated plate and at the top with a cap having a pipe connection. A layer of 2-inch crushed rock, 4 inches thick, placed on a perforated plate, represented the main body of the dam. Above this was placed a 10-inch layer of 1/2 inch to one inch gravel and a 14-inch layer of fine sand. Above the sand a 30-inch layer of compacted earth represented the impervious core. This impervious core was placed in layers and compacted by rodding equivalent to the compaction, which could be obtained by sheep's-foot rollers. Above the impervious section a reverse filter was placed, consisting of 4 inches of fine sand, 4 inches of 1/2 to one inch gravel, and 8 inches of crushed rock. Water under a head of 275 feet was admitted to the space at the top of the model and the performance of the impervious core and the filter noted. This model was kept under pressure for several months. The seepage, when it could be measured, checked the permeability of the core material as determined in the Soil Mechanics Laboratory. (When the air was dry, seepage evaporated and could not be measured.) No passage of the impervious material through the filter could be detected. When the model was dismantled after more than a year of operation, the impervious core was found to have been compacted into a dense hard mass, which contained little or no water. Other models were made and tested, with substantially the same result.

An earth deposit near the site, available for use as an impervious core, was tested and found satisfactory. All of the studies and tests indicated that an earth-faced rock-fill dam, built of materials readily available at the Nantahala site, would be a safe and economical structure.

The design of the earth-faced rock-fill dam for the Nantahala site proved to be relatively simple. (Figure 3.)


Downstream Edge of Crest
Max. El. 2918.87 As Built
Min. El. 2910
12.25'
1 on 1.4
1 on 1.4
1 on 2.5
Quarry Run Rock
Smaller Sizes
Intermediate Sizes
Larger Sizes
29.0'
Surface Under Earth Fill Excavated to Solid Rock Min. El. 2658.5
Surface Excavated to Firm Ledge Rock
0 40 80 120
Scale in Feet
Surface Stripped to Rock

(a) TYPICAL CROSS SECTION OF DAM


30'
13'
8'
3'
1 on 1.4
Tunnel Muck
Compacted Impervious Fill
Tunnel Muck
1 on 1.4
40'
Flood Surcharge
El. 2895
El. 2890 - Max. Normal Reservoir
Quarry Run Rock
3.3'
1 on 1.48
3" to 1"
5.04'
14.75'
30'0"
3" to 1"
Selected Rock 10" to 3"
38'6"
9'0"
11'0"
11'0"
25'0"
4'0"
6.5'
14.75'
Crushed Rock
Sand ¼ Down
Impervious Rolled Filled
1 on 1.37
Sand ¼ Down
Crushed Rock
1 on 1.37
Quarry Run Rock

(b) CREST DETAILS OF DAM

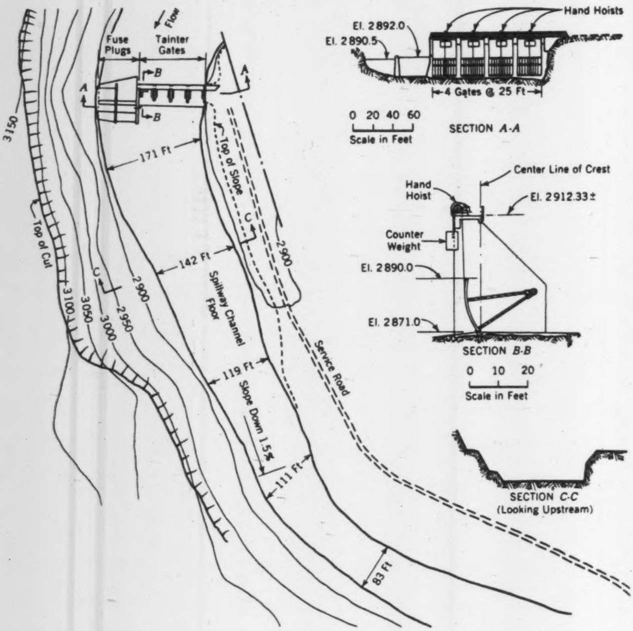
FIG. 3

Hydrology

The drainage area above the dam site is 91 square miles. It is mountainous, with steep forest-covered slopes. The average run-off is 331 c.f.s. It is believed that large, sharp crested floods, of short duration are possible, although there is no evidence of a flood exceeding 15,000 c.f.s. since the valley was settled 150 years ago. The full reservoir, with water surface at Elevation 2890 (N. P. & L. Co. Datum) has a surface area of 1,631 acres.

A diversion tunnel, 16 feet in diameter and 1,400 feet long, through the left abutment, together with an earth and rock-fill cofferdam, 50 feet high, diverted the river during construction. The diversion works were designed for a flood of 7,500 c.f.s., or about one half the record flood.

An adequate spillway is an essential feature of all dams. The Nantahala spillway (Figure 4) was designed as a channel, cut through the solid, rock of the right abutment, having a discharge capacity of 57,500 c.f.s., equivalent to a peak inflow of 88,000 c.f.s, utilizing a surcharge of 13 feet. The spillway will take care of a flood peak of 978 c.f.s. per square mile, without


Flow
Fuse Plugs
Tainter Gates
A
B
B
A
3150
171 Ft
Top of Slope
C
2900
Top of Cut
142 Ft
Spillway Channel Floor
C
3100
3050
3000
2950
2900
119 Ft
Service Road
Slope Down 1.5%
111 Ft
83 Ft
El. 2892.0
El. 2890.5
Hand Hoists
4 Gates @ 25 Ft
0 20 40 60
Scale in Feet
SECTION A-A
Center Line of Crest
Hand Hoist
El. 2912.33±
Counter Weight
El. 2890.0
El. 2871.0
SECTION B-B
0 10 20
Scale in Feet
SECTION C-C
(Looking Upstream)

PLAN AND DETAILS OF SPILLWAY

FIG. 4

encroaching on the 7 feet of freeboard above the surcharge level. (Figure 5.) The excavation from the spillway provided the rockfill for the dam. The spillway channel ends on a steep hillside, 400 feet downstream from the toe of the dam. The spillway discharge cascades into the original river channel.

The upper end of the spillway is closed by four hand operated Tainter gates, each 19 feet deep by 25 feet wide, and two sections of fuse plug between the end of the gate structure and the rock cut at the right of the spillway. The fuse plug adjacent to the Tainter gate structure is 33 feet 4 inches long, with its sill at Elevation 2871 and its crest at Elevation 2892, two feet above the crest of the Tainter gates. The fuse plug is composed of crushed rock, made watertight by a sand filter and earth blanket on the upstream slope. The earth blanket is protected against wave action by a filter and small rock cover. It is, in effect, a miniature earth-faced rock-fill dam, which will wash out when overtopped by more than one foot of water. The fuse plug adjacent to the rock cut is 25 feet long with its crest at Elevation 2890. 5. It is not intended to wash out when overtopped, but was provided primarily as a cushion to stop any rock which might spall off from the rock

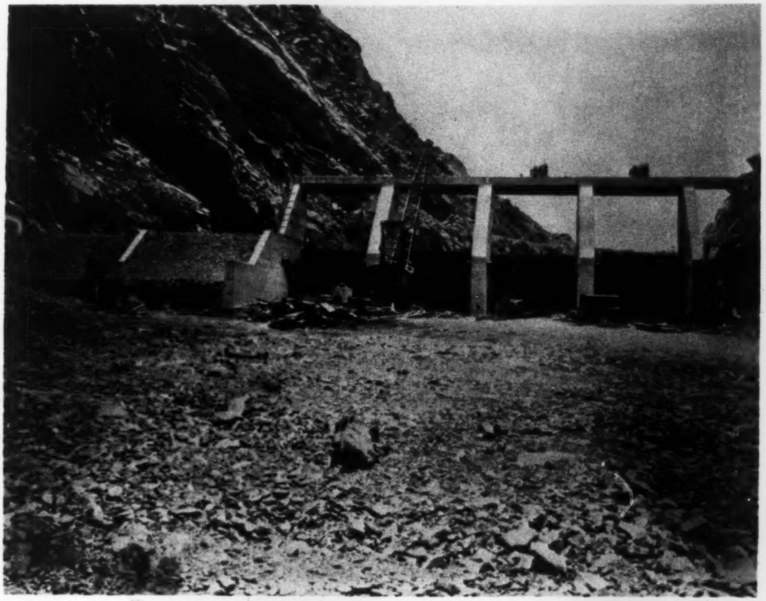


FIG. 5

face above it, without injury to the Tainter gates or fuse plug proper. The Tainter gates are counterbalanced and geared so that they can be opened or closed easily by one man. Hydraulic models of the spillway were made and tested. The final design was based on these tests.

The dam was designed as a triangular section rock-fill across the gorge, to be placed in one or more lifts as convenient for construction. (Figure 6.) Selective dumping placed the largest rock in the downstream part of the fill and the smallest quarry-run rock in the upstream part of the fill so that the voids in the main rock-fill would become progressively larger from the upstream face to the downstream face. Both the upstream and downstream slopes of the rock-fill were to be the natural angle of repose of the dumped rock. The first filter layer, 14.75 feet thick, was composed of selected rock not larger than 10 inches and not smaller than 3 inches. The second filter layer, 6.5 feet thick, was composed of crushed rock not larger than 3 inches and not smaller than 1/2 inch. The third filter layer, 6.5 feet thick, was composed of crushed rock fines smaller than 1/2 inch. The sizes specified and the grading of the filter materials produced by crushing fulfills the requirements that voids between the larger particles are filled by progressively smaller particles so as to prevent migration of the finer material. Theoretically, only a very thin filter layer would have been required. Actually, each

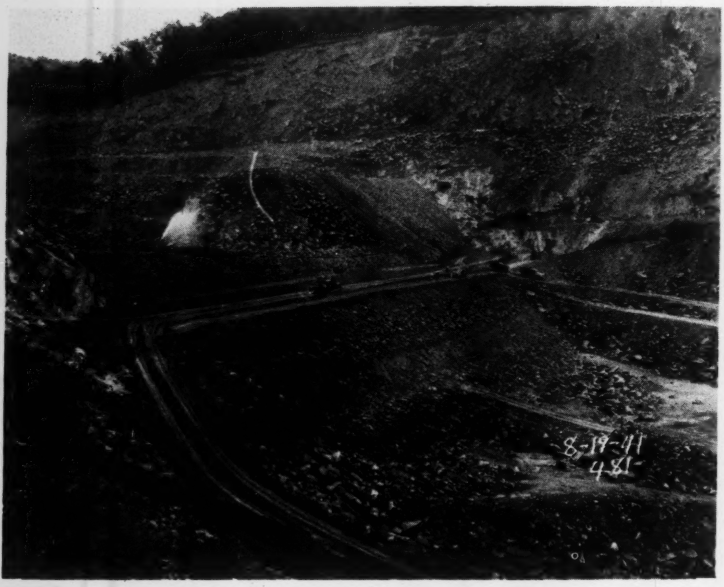


FIG. 6

filter was designed thick enough to be placed without difficulty and to adjust itself to settlement of the rock-fill without danger of rupture.

Only a small amount of material suitable for an impervious rolled fill was available near the site. This limited the rolled fill to a thickness required to prevent excessive seepage. Laboratory tests determined that an impervious rolled fill, 29 feet thick at the bottom, where the head is 230 feet, and 13 feet thick at the top, would have a seepage rate of not more than 2.75 cubic feet per minute, which was satisfactory. The rolled fill makes contact with the foundation in a cut-off trench excavated to solid rock and grouted. A 5 foot layer of crushed rock fines minus 1/2 inch and a second 5 foot layer of crushed rock 3 inches to 1/2 inch provides a reverse filter, above which was placed a layer of quarry-run rock to protect the rolled fill from wave action and the effect of sudden drawdown. The action of the rolled fill when subjected to the stress due to sudden drawdown and the ability of the upstream rock-fill to resist this stress was uncertain. To avoid these uncertainties, quarry-run rock was added until the core was safe against sliding, when analyzed by the circular arc method generally used to determine the stability of earth-fills made of homogeneous material. The resulting upstream slope is 1 on 2-1/2. Since the material above the rolled fill is free draining, the design is amply safe. The space between the diversion cofferdam and the main dam was filled with quarry waste, making the cofferdam an integral part of the main structure and increasing stability against sliding where drawdown stresses are the maximum. The upstream faces of the rock-fill, the filters, and the impervious rolled fill, are convex upstream so that as the dam settles the filters and the rolled fill will be further compacted. The top of the dam in the deepest section was cambered approximately 9 feet higher than the abutments to compensate for settlement.

Construction

After signing the contract in July 1940, the contractor began to assemble his forces and equipment and start work on clearing the dam site. To satisfactorily schedule the dam construction it was necessary to complete the river diversion as early as possible and most of the early work was devoted to the diversion tunnel. The contractor's forces began the portal excavation in September 1940 and had holed through in three and one half months. This was followed by construction of the concrete intake portal and placement of the concrete tunnel lining. Diversion was accomplished in March 1941.

As work progressed on the diversion structure, other forces completed the clearing and stripping of the dam site. Stripping was largely accomplished by ground sluicing using a total pump capacity of 8,400 gallons per minute at 275 feet head through light weight monitors. The larger materials which were not carried away by the river were excavated at the river's edge by power shovel and removed by truck. The entire foundation area occupied by the main rock-fill was stripped to rock. In the area covered by the impervious earth core, a cut-off trench was excavated to solid rock. Although it was necessary to excavate this trench 10 to 15 feet deep in one area, for the most part the cut-off trench excavation was limited to 2 to 6 feet. Upstream of the earth core, the foundation was only stripped of top soil and vegetation. Actually, however, there was little overburden remaining on the rock in this zone. In the cut-off trench area a grout seal was placed by drilling a series of percussion holes 25 feet deep and spaced on about 20-foot centers. These were followed by intermediate grout holes and further intermediate grout holes were required. In general, these holes were sealed at pressures ranging from 75 to 150 pounds per square inch. Following completion of this surface grouting, diamond core holes were drilled to a depth of 125 feet with holes spaced 100 feet apart in a single line near the center line of the cut-off trench. Many of these holes were drilled in an inclined position to intercept as many rock seams as possible. This curtain grouting was placed at 250 pounds per square inch pressure. Following completion at this spacing, intermediates were grouted until the holes were spaced 25 feet apart, and in some cases as close as 12-1/2 feet. Approximately 8,000 barrels of cement grout was required to make the foundation water-tight.

As soon as the diversion tunnel was ready for the river flow, a cofferdam was placed across the river channel and diversion accomplished. The cofferdam consisted principally of a rock-fill with an impervious blanket on the upstream slope, which later became part of the upstream toe of the main dam. With the river diverted, the balance of the stripping, grouting and foundation excavation was accomplished.

The spillway excavation was developed as a quarry and main source of rock for the dam. After stripping most of the earth and soft rock overburden, the hill was benched in at the dam crest elevation using wagon drills. When the bench was wide enough to form a shelf to hold the muck produced by coyote blasting, the shooting was done from coyote holes. After completing all excavation above this bench, work was started on the channel using wagon drills and excavating in benches approximately 20 feet deep until the channel grade was reached. The spillway layout was adjusted from time to time so that a balance could be established between quarry cut and embankment fill. The rock from the spillway was used in the main rockfill, end dumping from a series of high lifts averaging from 50 to 130 feet in height. In January 1941 the main downstream rock-fill was started beginning on the right abutment at a level 130 feet above the river bed. When the river was finally diverted it was necessary to get the fill over the left abutment as quickly as possible to prevent a delay in placement of the rolled earth-fill. This was accomplished by ramping down and reducing the height of the lift to 100 feet. Following this, the entire area was brought up to the 130-foot level and thereafter carried up in high uniform lifts across the entire width of the dam. The original dump slope of the rock was estimated to be one vertical to 1.4 horizontal and was thus shown on the contract drawings. However, actual observations during placement indicated that the rock was dumping to slopes of one vertical to between 1.30 and 1.35 horizontal. It was therefore necessary to regulate the placement of the rock lifts so that the actual placed downstream slope would approximate the slope shown on the drawings.

All quarry rock dumped on the fill was sluiced with water to wash the fines and chips into the voids of the larger rock. The sluicing plant provided 18 cubic feet of water per second under 275 feet head at the monitors. The ratio of water to rock-fill exceeded 4, which undoubtedly reduced the settlement which otherwise would have taken place. (Figure 7.)

The character of the rock and the methods of shooting produced quarry-run rock varying in size from 10 tons to dust. Normal quarry operation resulted in some trucks being loaded only with large rock, some loaded with small rock, and some loaded with a mixture of intermediate sizes. Selective dumping permitted placing most of the large rock in the downstream section of the fill and most of the small rock in the upstream section of the fill. This was accomplished without interfering with the quarry operation. Mention should be made of the compacting effect on the fill when dumping at the top of a high lift, particularly where the rock was large. It had been intended to smooth up the downstream slope but it was found impossible to move any of the rock without shooting. It was also found to be difficult and expensive to smooth up the slope by adding rock after completion of the fill. For these reasons the downstream slope of the fill was left exactly as placed. Some hand chinking was done on the upstream slope, particularly near the bottom where the voids were large. The surface of each lift was thoroughly scarified and sluiced to remove fines. (Figure 8.)

All filter materials were processed from small quarry rock and tunnel muck. Processed tunnel muck produced an excellent sand. Rigid laboratory control was exercised over the manufacture of the graded filter materials. Besides the usual grading, permeability and density tests, a high head permeability test was used to determine the suitability of the various filter materials for use in the dam. The pyralin walls of the permeameter permitted visual inspection of the filter materials for evidence of piping during testing. The minus 1/2 inch sand required the most careful control and a large share of the tests were confined to this material. The constant head

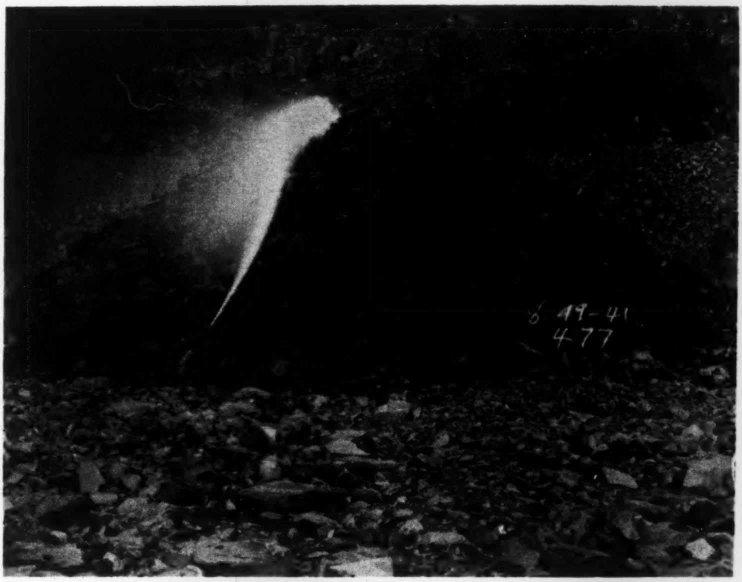


FIG. 7

type of test was used under gradients as high as 104. Besides visual inspection, which for several materials was test enough, a comparison of the grain size distribution before and after the test was made to study the extent of readjustments in the grading of the sand during the test. Records indicate that this constant control over processed filter material resulted in the construction of an effective transition from the fine soil particles to the large quarry-run rock.

The 3 to 10 inch filter material was first placed from the bottom of the dam just several feet ahead of the other filter materials, but, as considerable interference developed in placement, the balance of this material was dumped down for heights of approximately 100 feet from the main rock-fill without undue segregation. The 1/2 to 3 inch and the minus 1/2 inch materials were placed from the bottom using 5-yard dumptors. The sand filter was always placed at a level slightly below the 1/2 to 3 inch rock to preserve continuity of the filters. Both materials were sluiced, spread and compacted with dozers and rollers. As it was necessary to cross the rolled earth-fill in placing these filter materials, considerable care had to be exercised to prevent tracking the earth onto the filter materials and thereby reducing their effectiveness. These filter materials could also be placed to advantage by dumping from the top of the main rock-fill through chutes.

FIG. 8

Before any rolled earth-fill was placed, the bottom of the cut-off trench was scrubbed clean with compressed air and water. One borrow pit provided all earth material incorporated in the dam. This was developed as a scraper pit using four carry-all scrapers of from 16 to 20 cubic yards capacity hauled by RD-8 tractors. The earth-fill was placed in the core using bull-dozers to grade and spread the material. Compaction was attained with two sheep's foot rollers mounted side by side, with 176 tamping feet, each having an area of 5.4 square inches and developing a pressure of 293 pounds per square inch when fully loaded. In areas where the roller was inaccessible, the fill was compacted by power tampers. Additional compaction was obtained by the hauling and placing equipment which was not permitted to track. The fill was placed in 6-inch horizontal layers just slightly behind the placement of the downstream filter materials.

For purposes of control and before placing any earth-fill, a test fill was constructed to study compaction of the soil under various combinations of rolling equipment. A study of the test fill results was the basis of the placement criteria developed for the dam fill. Regular periodic sampling was started and laboratory tests performed to check the results against the criteria.

Originally it was planned to require the impervious core to be compacted to a density of 98 pounds per cubic foot dry weight as determined by the Proctor test. At this density, the optimum water content averaged 22.3 per cent. After rolled fill operations were started it was evident that strict adherence to the optimum water content would be impossible. The natural water content of the borrow pit averaged 5 per cent higher than the optimum value and there was no opportunity to dry it because of the excessive rainfall in the area.

A study was made to determine how rapidly the core would consolidate, using soil characteristics obtained from the consolidation test, and assuming the material was placed at a moisture content greater than the optimum value. These studies indicated that by the time construction was completed the earth core would be fully consolidated under the superimposed loads of the upstream rock-fill and filters, no matter if the material was placed wet. With this in mind, the following criteria were developed as compaction requirements for the impervious rolled fill:

1) The minimum compaction should be that which is required by the static state (no drawdown). Under this condition, the required average unit dry density was 94.0 pounds per cubic foot. Dry weights less than 90.0 pounds per cubic foot were limited to one per cent of the samples tested.

2) The minimum water content was limited to 2 per cent less than the optimum value as determined from the Proctor test.

3) The maximum water content was limited by the ability of the hauling equipment to operate without miring.

Placing the material on the wet side of the optimum water content had the advantages of low air voids and a flexible core which could readily adjust itself to the initial readjustments that take place in the main rockfill.

Three methods of control sampling were used on the impervious core:

1. Moisture samples were taken every two hours during the placement of fill to check the water content.

2. Control cylinders were taken at regular intervals to check the density of the fill against that required by the compaction criteria. These cylinders were 5 inches in diameter and 5 inches high, and were taken with a piece of thin walled pipe with a sharpened end. Samples were usually taken at 1-1/2 and 3 feet depths below the surface to avoid disturbance by the sheep's foot roller and hauling equipment. Since the cylinders were calibrated as to weight and volume, the density of the sample was readily determined by weighing the full cylinder and determining the moisture content.

3. Chunk samples were taken at regular intervals to check all physical properties of the fill. This type of sampling required large test pits and interferred somewhat with placement operations. For this reason, tests of this type were held to a minimum. It was the most accurate method of sampling and was therefore used for record testing. The chunks were approximately one cubic foot in size and were taken from the fill at a depth of 3 feet and sometimes 5 feet to obviate any disturbance due to hauling and rolling equipment.

Routine control sampling and testing was performed continuously with the placement of the impervious rolled fill. Moisture content results were generally available within two hours but, where needed, results were obtained in 20 minutes if a volumetric method was used. One control cylinder was taken on the average with every 1200 cubic yards of earth placed. One chunk sample was taken on the average with every 2800 cubic yards of earth placed.

Chunk samples were subjected to a mass gravity test for density determination, specific gravity test, permeability test, mechanical analysis, shear and consolidation tests. Figures 9 to 13 indicate the average results of such tests. Bag samples were also taken adjacent to each cylinder and chunk sample and these were given a compaction test for comparative purposes in studying the degree of compaction given the rolled earth-fill.

The density results of all chunk samples have been plotted in graph form, including wet and dry unit weights, water content, void ratio and per cent air voids. (Figure 14.) Placement of the rolled earth-fill began in June 1941 and was completed in November 1941.

The upstream filters were placed along with the impervious rolled earth core. A stability study of the upstream slope indicated that it was permissible to let the placement of the upstream rock-fill lag as much as 50 feet behind the placement of the rolled earth-fill without any possibility of an earth slide. Rock-fill operations were therefore concentrated on completing the main downstream rock-fill so as not to hold up the rolled earth-fill operations. After the downstream rock-fill had gained considerable lead over the rest of the operations, the upstream rock-fill was started. The upstream rock was dumped in shallow lifts of 10 to 15 feet high, using two dozers to bring the rock to the design slope of 1 on 2-1/2. Most of the rock going into this zone of the dam came from a low level in the spillway quarry and was relatively clean of dirt and dust. Sluicing was done with a monitor fed by a 1500 gallon per minute pump.

Because of the limited working space at the top of the dam, the various zones of material in the upper 40 feet of the embankment were brought up to the crest elevation simultaneously, but the same methods and care in placement were employed. The dam was completed in February 1942, 11 months after the site had been unwatered. The following quantities of material were used:

Quarry-run rock	1,692,000 cubic yards.
Process rock	350,000 cubic yards.
Rolled earth-fill	223,000 cubic yards.
Total quantity measured in the structure	2,265,000 cubic yards.

A view of the completed dam, with the water surface at Elevation 2858, taken October 2, 1942, is shown in Figure 15.

Performance

The Nantahala dam has been in operation nine years. The reservoir has been substantially filled and emptied each season. Careful inspection has failed to disclose any evidence of local settlement or the displacement of rock. The spillway has been used three times.

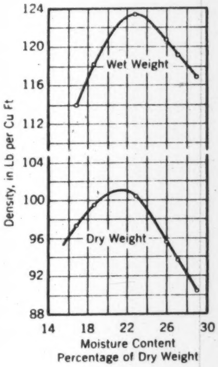


FIG. 9

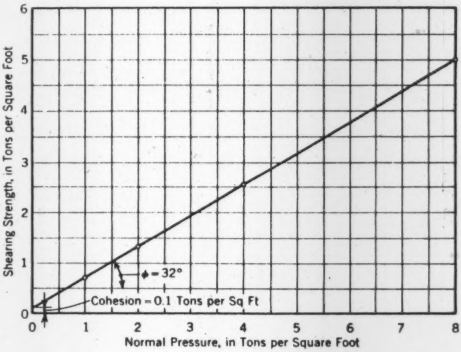


FIG. 10

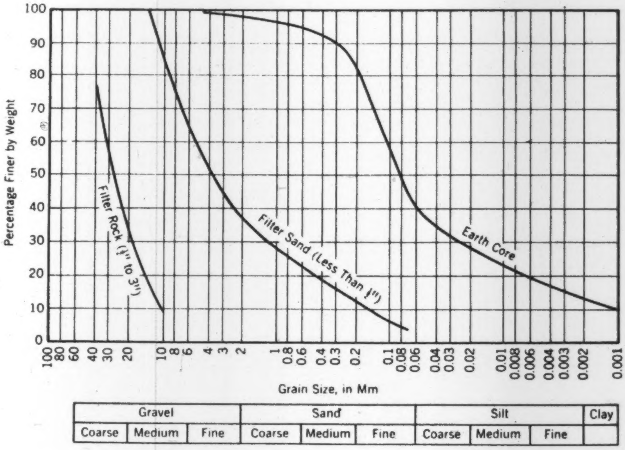


FIG. 11

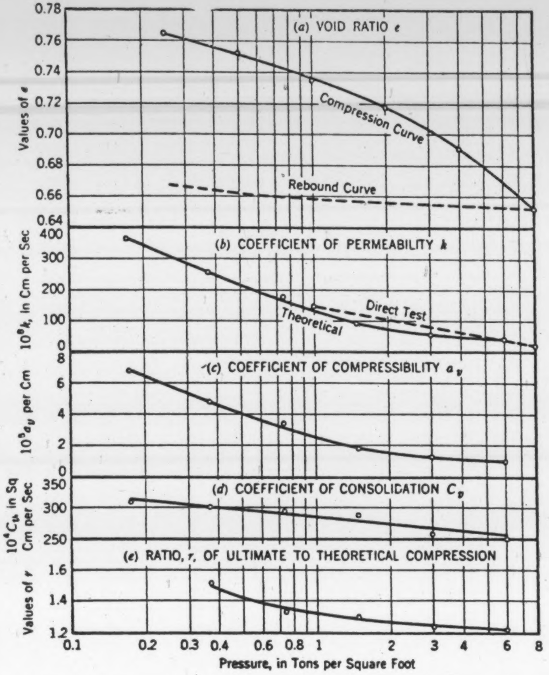

(a) VOID RATIO e
Values of e
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0.64
Compression Curve
Rebound Curve
(b) COEFFICIENT OF PERMEABILITY k
$10^8 k$, in Cm per Sec
400
300
200
100
0
Direct Test
Theoretical
(c) COEFFICIENT OF COMPRESSIBILITY a_v
$10^3 a_v$ per Cm
8
6
4
2
0
(d) COEFFICIENT OF CONSOLIDATION C_v
$10^4 C_v$ in Sq Cm per Sec
350
300
250
(e) RATIO, r, OF ULTIMATE TO THEORETICAL COMPRESSION
Values of r
1.6
1.4
1.2
0.1
0.2
0.3
0.4
0.6
0.8
1
2
3
4
6
8
Pressure, in Tons per Square Foot

FIG. 12

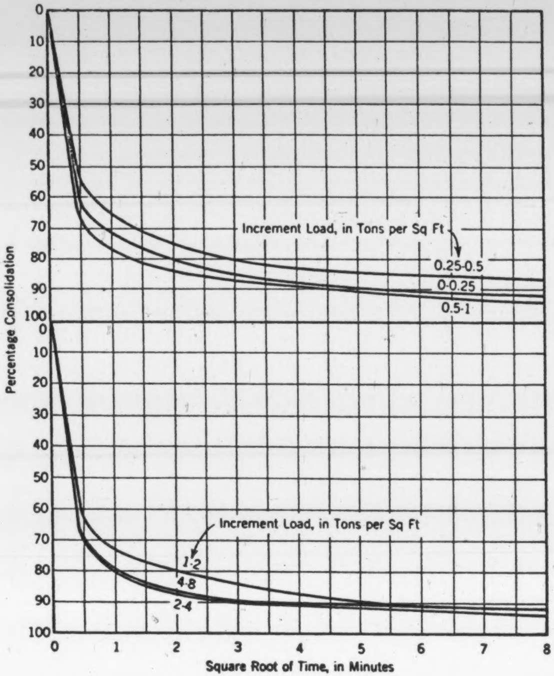

Percentage Consolidation
0
10
20
30
40
50
60
70
80
90
100
Increment Load, in Tons per Sq Ft
0.25-0.5
0-0.25
0.5-1
Increment Load, in Tons per Sq Ft
1-2
4-8
2-4
0
1
2
3
4
5
6
7
8
Square Root of Time, in Minutes

FIG. 13

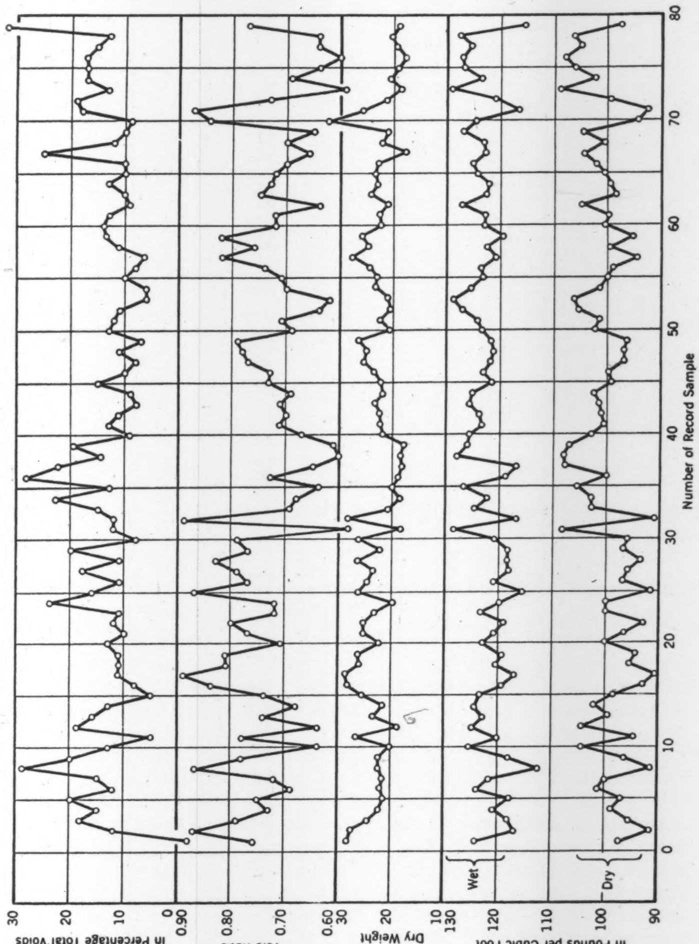

Air Voids, in Percentage Total Voids
30
20
10
0
Void Ratio
0.90
0.80
0.70
0.60
Water Content, in Percentage Dry Weight
30
20
10
Unit Weight, in Pounds per Cubic Foot
130
120
110
100
90
Wet
Dry
Number of Record Sample
0
10
20
30
40
50
60
70
80

FIG. 14

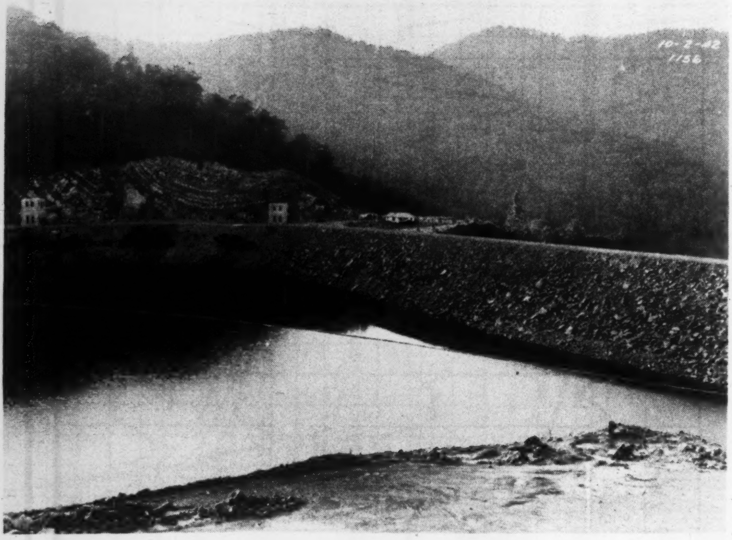
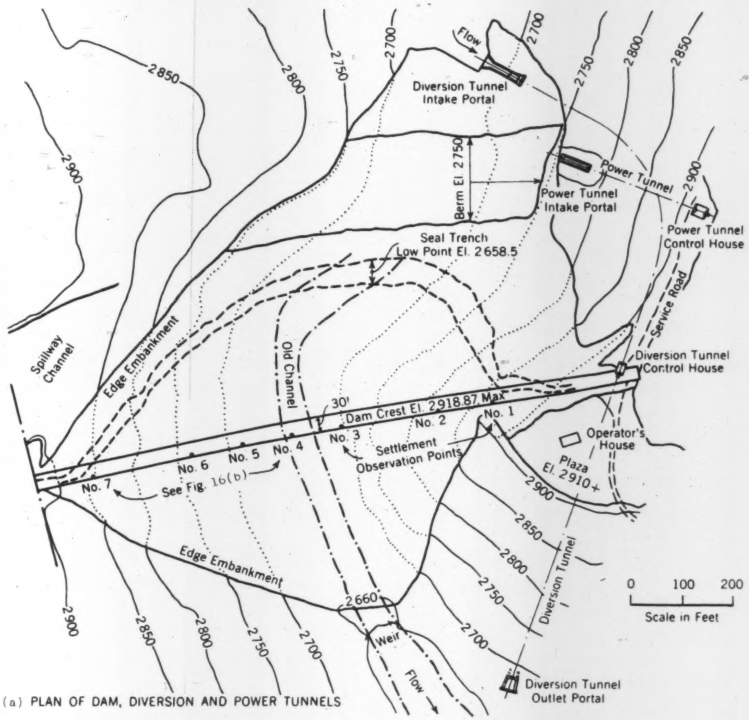


FIG. 15

The movement of the crest of the dam, vertically and horizontally, has been measured each month. These measurements, including December 1946, are shown graphically on Figure 16, from which you can see that the settlement has been greatest in the highest section of the dam, decreasing to zero at the abutments. During the year 1947 the vertical settlement at the highest section was 0.19 feet and the horizontal movement downstream was 0.10 feet. Since January 30, 1942, when the dam was closed, the maximum vertical settlement has been 2.16 feet (less than 1 per cent of the height of the fill) and the maximum downstream movement has been 1.01 feet.

A small weir was built across the old channel below the dam so that the flow past the dam could be measured. The flow measured at the weir includes the surface run-off from an area of 12 acres, the flow of two small springs in the abutments downstream from the grout curtain, and the seepage from the dam. For this reason the measured flow varied widely. It is shown graphically on Figure 17. The actual seepage through the dam cannot be definitely determined, but it is believed to be substantially equivalent to the computed seepage (2.75 cubic feet per minute), based on permeability tests of the rolled fill. In any event, it is very small.

Water was released through the spillway for 14 days in May 1944 for 55 days in January, February, and March, 1946 and for several days in 1950.


Flow
Diversion Tunnel
Intake Portal
2850
2800
2750
2700
2900
Berm El. 2750
Power Tunnel
Power Tunnel
Intake Portal
Power Tunnel
Control House
Seal Trench
Low Point El. 2658.5
Service Road
Edge Embankment
Old Channel
Spillway
Channel
Diversion Tunnel
Control House
30'
Dam Crest El. 2918.87 Max
No. 1
No. 2
No. 3
No. 4
No. 5
No. 6
No. 7
Settlement
Observation Points
See Fig. 16(b)
Operator's
House
Plaza
El. 2910+
Edge Embankment
2660
Weir
Flow
Diversion Tunnel
0 100 200
Scale in Feet
Diversion Tunnel
Outlet Portal

(a) PLAN OF DAM, DIVERSION AND POWER TUNNELS

FIG. 16(a)

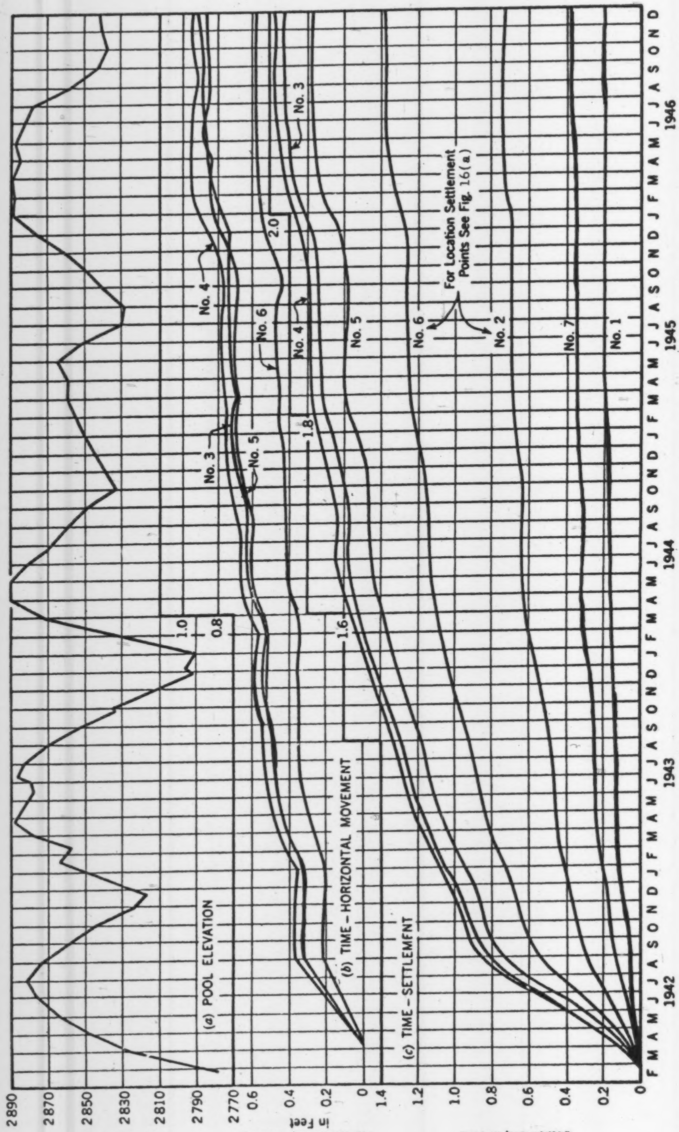

Pool Elevation
2890
2870
2850
2830
2810
2790
2770
Horizontal Movement, in Feet
0.6
0.4
0.2
0
1.0
0.8
1.6
1.8
2.0
Settlement, in Feet
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
(a) POOL ELEVATION
(b) TIME - HORIZONTAL MOVEMENT
(c) TIME - SETTLEMFNT
No. 3
No. 5
No. 4
No. 6
No. 4
No. 3
No. 5
No. 6
No. 2
No. 7
No. 1
For Location Settlement Points See Fig. 16(a)
F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
1942
1943
1944
1945
1946

FIG. 16(b)

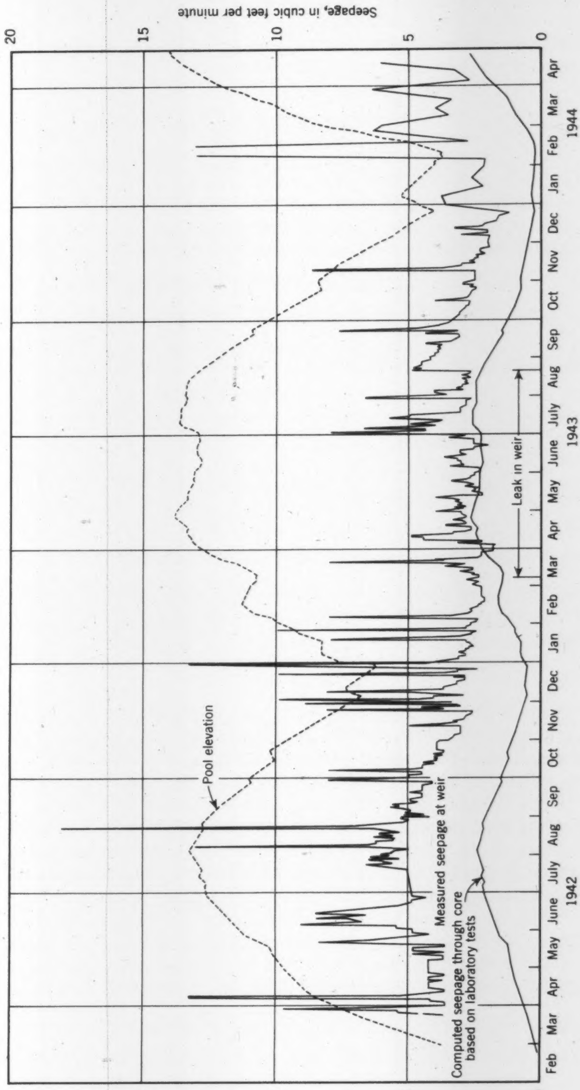

Pool elevation, in feet
2750
2800
2850
3000
3500
Seepage, in cubic feet per minute
0
5
10
15
20
Pool elevation
Measured seepage at weir
Computed seepage through core based on laboratory tests
Leak in weir
Feb
Mar
Apr
May
June
July
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
May
June
July
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
1942
1943
1944

FIG. 17

The maximum rate of release was 5,000 c.f.s. The water released through the spillway washed a substantial quantity of loose material from the hillside into the old river channel, as it was expected to do. The debris made a second dam which submerged the measuring weir.

CONCLUSIONS

The Nantahala rock-fill dam with an impervious earth section on the upstream slope as designed and built, has functioned satisfactorily, with every indication that it will continue to do so for an indefinitely long time.

The following comments appear to be warranted:

1) The materials from which the dam is built will continue to disintegrate, but at a very slow rate, measured in geological rather than in historical time.

2) Settlement of the rock-fill will continue indefinitely at a decreasing rate.

3) The water load is applied normal to the inclined impervious section of the dam. All loads are transmitted directly to the foundation; the heaviest load through the shortest distance.

4) No hydrostatic pressure can be built up in the interior of the dam.

5) The structure is substantially water-tight. The impervious section of the dam adjusts itself to settlement in the rock-fill without rupture.

6) The dam was constructed of materials readily available at the site. The design facilitated construction and no serious difficulties were encountered.

ACKNOWLEDGMENTS

The dam was designed by and constructed under the supervision of the Hydraulic Engineering Department of the Aluminum Company of America. It was built by the Utah Construction Company within the time originally specified in the contract.

Doctor Warren J. Mead, Mr. Joel D. Justin, Mr. William P. Creager, Doctor Glennon Gilboy, and Mr. Robert R. Philippe served as Consulting Engineers during the design and construction of the entire project. Their services contributed immeasurably to its success.

DISCUSSION

F. L. LAWTON,[2] F. ASCE.—The Nantahala Dam owes its inception to Col. Growdon's interpretation of the genesis of many glacial lakes, formed by closure of old stream beds by glacial moraines. The conversion of Nature's method of construction to Man's has resulted in the development of a type of dam applicable to difficult sites, having an inherently indefinitely long life, and one which frequently can be constructed at a far cheaper cost than other types.

The settlement data reflects the variation in reservoir level as is the case with many other rockfill dams. It would be interesting if the author could indicate whether there has been any signs of lateral movement of the fill, i.e. from the abutments to the maximum section.

WALDO G. BOWMAN,[3] F. ASCE.--The sloping core rockfill dam certainly deserves to be classed as a new type. deserves to be classed as a new type.

Concrete dams can be classified or differentiated one from another according to the structural action involved—gravity, arch, buttress, etc. For rockfills, however, in which the structural action is always of gravity type, some other means of classification must be adopted, and the kind of water barrier used is a logical choice.

Using this means of classification, there are perhaps only four basic types of rockfill dams—rigid central barrier, central earth core, rigid upstream face and flexible upstream face. A further refinement of this classification could be made by taking into account the material of which the barriers are constructed, but the dam type would be essentially the same.

Whether, for example, a rigid upstream face is made of concrete, steel or wood is immaterial so far as dam type is concerned. The same would be true for a flexible face of either asphalt or plastic sheets—if the latter should ever be tried. But the sloping earth core, while basically a flexible upstream facing, introduces a distinctive new principle to water barrier design. This principle changes the character of the dam and, in effect, creates a new type.

The principle involved is, of course, to sandwich a thin sheet of earth between rock filter layers which are graded outward from fine to coarse on both

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2. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.
3. Editor, Engineering News-Record, New York, N. Y.

sides of the earth. The downstream filter thus keeps the earth barrier, even though it deforms, from migrating through the dam, while the reverse filter upstream performs a similar function in case of sudden drawdown, while it also protects the earth from wave action. The sloping earth core thus performs its function and maintains its structural integrity by quite different means than do other types of flexible barriers. Beyond this fact it is fashioned from materials ready at hand, which can be a decided advantage.

It seems proper to emphasize the uniquely different character of this flexible upstream barrier, even though its significance will not be missed by many readers of the paper.

There is a second significant aspect of the sloping earth core dams described in this paper. It is that in the short space of a decade a new type dam was proposed, designed, built and followed by the construction of six others. Few new developments in any branch of civil engineering have enjoyed such unusual initial success. And it is particularly notable that the same owner, the Aluminum Company of America, which would not be likely to repeat a design just out of pride in the fact that it originated it, is responsible for six of the dams.

This paper gives a remarkably complete record of the sloping earth core dam from inception through design, construction and operating performance. As such, it constitutes a valuable addition to engineering knowledge for which the profession owes the author its gratitude. It might conceivably demonstrate this gratitude by referring to such structures as rockfill dams of the Growdon sloping earth core type.

JAMES P. GROWDON,[4] F. ASCE.—In his discussion, Mr. Bowman points out, with great clarity, the essential differences between the sloping earth core rockfill dams and other types. The suggestions he offers are most flattering to the writer.

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[4] Consultant, Aluminum Co. of America, Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3067

ROCKFILL DAMS: MUD MOUNTAIN DAM

By Allen S. Cary,[1] M. ASCE

With Discussion by Messrs. J. B. Cooke; F. L. Lawton; and Allen S. Cary

SYNOPSIS

Mud Mountain Dam, White River, Washington, an earth and rock fill structure 400 feet high, has settled in the core zone almost exactly as predicted during design studies which were based on large scale consolidation tests. Differential settlement between core and shell zones caused cracking at the crest parallel to the axis of the dam with rock shells settling about 18 inches more than the core.

INTRODUCTION

Mud Mountain Dam is a flood control dam constructed 1939 to 1941 by the Seattle District Corps of Engineers on the White River, Washington near Enumclaw, 50 miles southeast of Seattle. White River drains the glaciers on the northeast and north sides of Mount Rainier. It is a swift mountain stream falling 400 feet in the seven mile length of reservoir above the dam.

Site

The dam site is a narrow rock canyon with the right side at the axis rising vertically from elevation 870 to 1090 and the left side sloping about 1/3 on 1 from 870 to 1150. Above the top of rock at elevation 1130, extremely compact glacial till occurred which sloped at about 1 1/2 on 1 to elevation about 1300. The till continues to elevation about 1250 and is overlain by glacial lake silt and clay through which the spillway channel was excavated on the right bank.

Note.—Published essentially as printed here in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1745. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Geol., Soils Lab., U. S. Engrg. Dept., Univ. of Washington, Seattle, Wash.

Structure

Mud Mountain Dam is a rock fill structure 400 feet high with a central rolled earth core. Slopes are symmetrical up and downstream about the central axis, the top 150 feet of the dam having 1 on 1-3/4 slopes and the lower 250 feet having 1 on 2-1/4 slopes. The central core is 30 feet thick at the top and 150 feet thick at the base. Transition zones on either side of the core are each 10 feet thick at the top and 50 feet thick at the base.

Materials

Materials used in the dam were:

a. Rock fill, an andesite tuffaceous to very hard.

b. Transition, crushed fine grained granite.

c. Core, pit-run sand and gravel with an admixture of 20 percent silty glacial till-like material to reduce permeability. All cobbles over 6 inches were removed from the core material.

Construction

During construction of the dam all riverbed materials were removed to the surface of bedrock under the entire dam, a distance of about 1600 feet along the stream bed. Canyon walls were sloped to insure wedging action as the fill settled and to eliminate overhangs in the rock. The core zone was constructed in 6-inch compacted layers and the transition zones in 12-inch layers. Rock fill was dumped in 40-foot lifts and sluiced with 1-1/2 cubic yards of water per cubic yard of rock.

Settlement Gages

In order to measure the settlement of the dam, gages were installed along the axis of the dam in the core as construction proceeded at 100,200 and 300 feet above the bedrock surface. Monuments were also installed along the crest of the dam as well as at 50-foot intervals on the upstream and downstream slopes of the rock shell zones.

Settlement

Settlement computations based on large scale consolidation tests indicated that the maximum settlement of the core section following completion of the dam would be 2.9 feet. The crest was overbuilt a maximum of 4 feet or 1 percent of the height of fill along the axis. Settlement measurements from the date of completion of the dam in 1941 continuing until 1950 indicated a total maximum settlement of 2.7 feet at the crest of the dam over the deepest canyon section, 400 feet. Thus it can be seen that actual settlement was very close to the theoretical in 1950. No further measurements have been taken. The monuments which were placed at 50-foot intervals on the rock fill slopes were checked for horizontal movement until 1950. Horizontal movement varied from zero to a maximum of 0.90 foot. No advance predictions had been made on this movement.

Cracks Along Top of the Dam Between Core and Shell

In August 1942 cracks appeared on the crest of the dam along the junction between the core and the upstream and downstream transition zones parallel to the axis. Test pits revealed that the cracks were completely indiscernible at 6 feet depth. The cracks were excavated and backfilled but continued movement through 1948 reopened the cracks to a maximum of about 4 inches wide. The shell zones had settled a total of about 18 inches below the core zones. The differential settlement between core and rock shell zones was not considered serious and since 1948 there has been no measurable differential movement.

CONCLUSIONS

The maximum pool level which has been reached in flood control operation has been 1117 feet — 98 feet below spillway crest of 1215 feet. No study of the effect of full pool on behavior of the dam has been made and none seems warranted in view of the close parallel between predicted and actual settlement. It is probably that the settlement of the rock shells could have been reduced if more sluicing water had been used and also if the rock had all been very hard such as basalt instead of some tuffaceous andesite.

DISCUSSION

J. B. COOKE,[2] F. ASCE.—The author gives an interesting summary of the high central core rockfill dam. Additional detail on the settlement would be of particular value, since measurements as described by the author, but briefly summarized, are seldom as thoroughly taken.

The profile at Mud Mountain puts the lower half of the dam between steep canyon walls and the abutments are more gradually sloped for the upper half of the dam. A profile of the vertical settlement plotted over this abrupt bedrock profile would be of particular interest. For example, does the crest over the top of the nearly vertical canyon wall settle as a 200-foot dam or as a 400-foot dam, and is there any arching tendency over the narrow canyon that might reduce settlement over that which would occur in a broad 400-foot fill. It has been the writer's observation that the crest settlement profile is much smoother than the foundation profile and, for more gradual profiles than that of Mud Mountain, affected more by the high central fill than the shallower abutment fill. The maximum movement at the highest section seems to drag the adjacent sections of less height.

The cracks between core and shell have been observed on a number of dams and are not very deep. They are generally agreed to be of little concern. An interesting reference on such cracks, aside from this Symposium, is an article by Raul P. Marsal and Enrique Tamez in the Indian Journal of Power and River Development, Vol. VII, No. 5, May 1957, p 18, 5 pp, 2 ff, 2 plates. Cracks which did not occur at Mud Mountain, but which would cause concern in a core rockfill dam would be transverse to the core and in the core near the ends of the crest of the dam. The tendency, particularly of the rockfill, is to develop tension near the abutments and compression near the center. This is most evident in impervious face rockfills and should be less so in sloping and then in central core rockfills as the influence of the rockfill decreases. The tensions may not cause cracks and, if they should, the filters are there to insure healing. However, as rockfill dams become higher, the predictions of movements becomes increasingly important. Few lateral movement measurements on core rockfill dams have been published and those taken on the high Mud Mountain would be valuable.

The downstream movement of crest points would be of interest even though the dam has been filled to only 300 feet of its 400-foot height.

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[2] Superv. Civ. Engr., Pacific Gas and Electric Company, San Francisco, Calif.

F. L. LAWTON,[3] **F. ASCE.**—Despite the fact the settlement in the Mud Mountain Dam was about as anticipated, it would appear rather surprising that shortly after completion of the dam cracks appeared on the crest along the junction between the core and the upstream and downstream transition zones parallel to the axis. It is true the cracks are reported as having been ". . . completely undiscernible at 6 feet depth", but this is not necessarily an indication of no slippage of the shell zones on the transition zones. Would the author care to comment on this point?

It is noted that the rockfill consists of an andesite described as being "tuffaceous to very hard". How has this rockfill withstood weathering?

ALLEN S. CARY,[4] **M. ASCE.**—The discussions and questions raised by Messrs. Cooke and Lawton are pertinent and should be completely answerable from the record. Unfortunately, the records are not as complete as the author originally believed. It has come to light that the monuments on the top of the dam along the axis and on the slopes were not set for a period of 70 days after completion of the dam. The statement that the 1950 measurement indicated a total settlement of 2.7 ft is, therefore, incorrect.

Fig. 1 shows a section of the dam along the axis with the measured settlement of the gages at El. 950, 1050, and 1150 ft plotted at an exaggerated scale.

Fig. 2 shows, graphically, the settlement measurements on all gages. Readings taken from 1947 to 1950 indicate practically no movement.

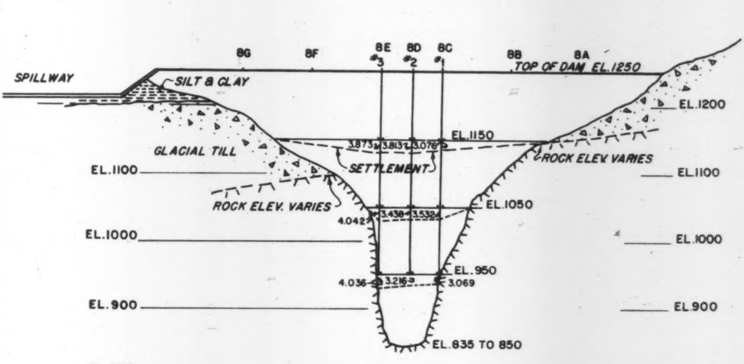
8G
8F
8E #3
8D #2
8C #1
8B
8A
SPILLWAY
SILT & CLAY
TOP OF DAM EL.1250
EL.1200
GLACIAL TILL
EL.1150
3.873
3.813
3.076
SETTLEMENT
ROCK ELEV. VARIES
EL.1100
EL.1100
ROCK ELEV. VARIES
EL.1050
4.042
3.438
3.532
EL.1000
EL.1000
EL.950
4.036
3.216
3.069
EL.900
EL.900
EL.835 TO 850
EL.800
EL.800

FIG. 1.—SECTION ALONG AXIS OF DAM, LOOKING UPSTREAM

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[3] Chf. Engr., Power Dept., Aluminium Labs., Ltd., Montreal, Canada.

[4] Geol., Soils Lab., U. S. Engrg. Dept., Univ. of Washington, Seattle, Wash.

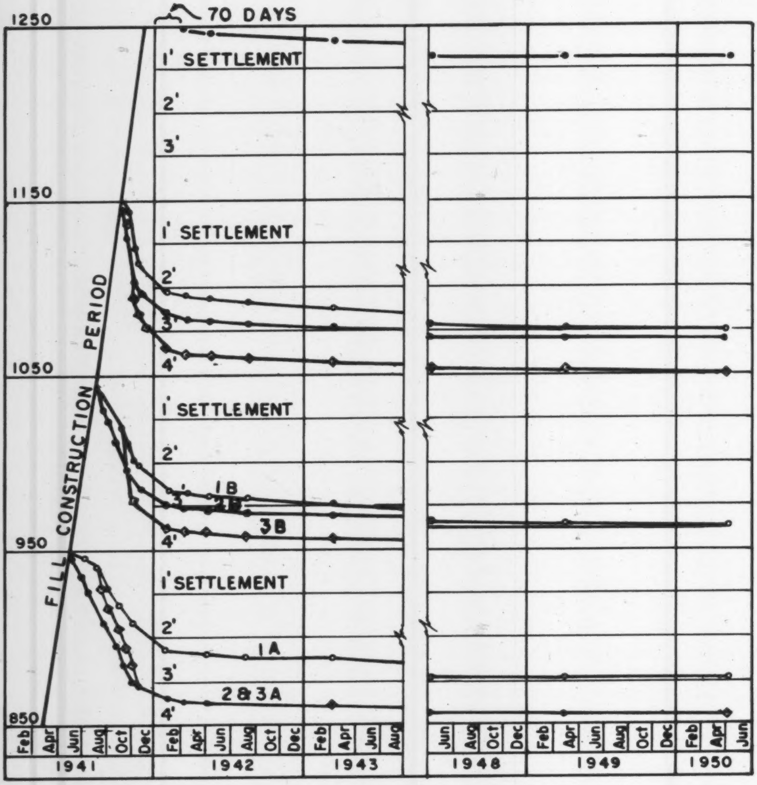
70 DAYS
1250
1150
1050
950
850
1' SETTLEMENT
2'
3'
4'
FILL CONSTRUCTION PERIOD
1B
2B
3B
1A
2&3A
Feb
Apr
Jun
Aug
Oct
Dec
1941
1942
1943
1948
1949
1950

FIG. 2

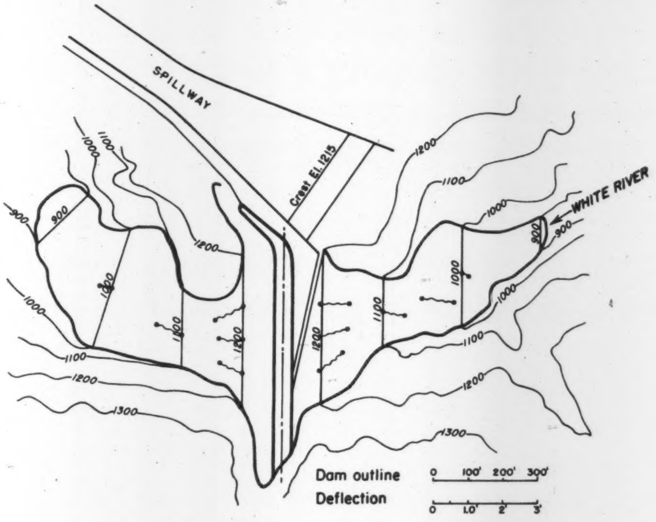
SPILLWAY
Crest El. 1215
WHITE RIVER
900
1000
1100
1200
1300
Dam outline
0 100' 200' 300'
Deflection
0 1.0' 2' 3'

FIG. 3.—LATERAL DEFLECTION OF MONUMENTS TO MAY 1950

Horizontal movement of monuments on the top of the dam set 70 days after completion along the axis indicate lateral movements by 1949 as follows:

Monument No.	Downstream Component	Upstream Component
8A	0.02 feet	
8B	0.08 feet	
8C	0.11 feet	
8D	0.11 feet	
8E	0.05 feet	
8F		0.20 feet
8G		0.15 feet

The upstream and downstream components of movements are attributed entirely to topography of the canyon walls. Fig. 3 indicates the horizontal component of the downslope movement of monuments set on the slopes.

A great percentage of the rock is hard and brittle showing no evidence of weathering. A small percentage of the tuffaceous andesite has weathered severely, but the total volume of this rock is so small that it can affect the total settlement of rock shells very little.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3068

ROCKFILL DAMS: TVA CENTRAL CORE DAMS

By George K. Leonard,[1] F. ASCE and Oliver H. Raine,[2] M. ASCE

With Discussion by Mr. F. L. Lawton

SYNOPSIS

Especially valuable are reports on the behavior of rock fill dams to compare with the design assumptions. The many variable and sometimes unknown conditions which influence the design of fill dams are sometimes either over- or under-emphasized and, therefore, misapplied. Only by comparing the results with the design assumptions can better dams be built. The authors have made this comparison on three TVA fill dams.

INTRODUCTION

The central core rock fill dams as built by the Tennessee Valley Authority (Fig. A) naturally differ from those built by other owners due to varying design assumptions, available materials, and methods of construction; but generally speaking, all consist of a heavy, nearly impervious central earth core supported and stabilized by massive dumped rock fills on the upstream and downstream faces. While emphasis will be on the performance of three of TVA's dams beginning with reservoir impoundment, it seems necessary and appropriate to describe in some detail their planning, design, and construction.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1736. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Cons. Engr., Knoxville, Tenn.

[2] Civ. Engr., Tennessee Valley Authority, Knoxville, Tenn.

Advantages of This Type of Dam

Central core rock fill dams (Fig. 1), if properly chosen, designed, and built, have many excellent qualities. Most important of these might be their durability, their permanence. Neither fire nor flood, heat nor cold affect their life in any way. In time of war, they would have a much better chance for survival than concrete dams. They are as permanent as the hills into which they abut.

Dams of this type are economical in that use is made of natural materials at the site instead of the more costly materials required in a concrete structure; design is less complicated; and construction is relatively simple.

Three of TVA's Rock Fill Dams

Three central core rock fill dams built by the Tennessee Valley Authority will be described. These are all located on the upper tributaries of the main river, Watauga and South Holston in the northeast corner of the watershed and Nottely in the southeast corner, all on rivers of the same name. The three projects serve multipurpose objectives and have considerable flood control storage as well as power installations. A 15,000-kw unit has recently been installed at Nottely to utilize the available energy from the discharge to the downstream Hiwassee project during peak operation. Fig. 1 shows cross sections of the three dams, and Figs. 7, 8 and 9 show views of the dams.

Table of Principal Features

	Nottely	Watauga	South Holston
Time of Construction	1941-1942	1947-1948	1948-1950
Crest Length - Feet	1,942	900	1,600
Maximum Height - Feet	184	318	285
Camber for Settlement - Feet	1.7	9.2	10.1
Core Slope - Upstream	1 on 1	1 on .85	1 on .75
- Downstream	1 on 1.2	1 on .85	1 on .75
Face Slope - Upstream	1 on 2	1 on 2	1 on 2.2 - 1 on 2.1
- Downstream	1 on 2	1 on 2	1 on 2
Quantities - Millions of Cu. Yds.			
Rolled Earth Fill	0.85	1.48	2.09
Rock Fill*	0.70	2.02	3.81
Nominal Top Elevation	1794	1998	1765
Maximum Design Headwater	1788	1988	1755
Normal Maximum Headwater	1780	1959	1729

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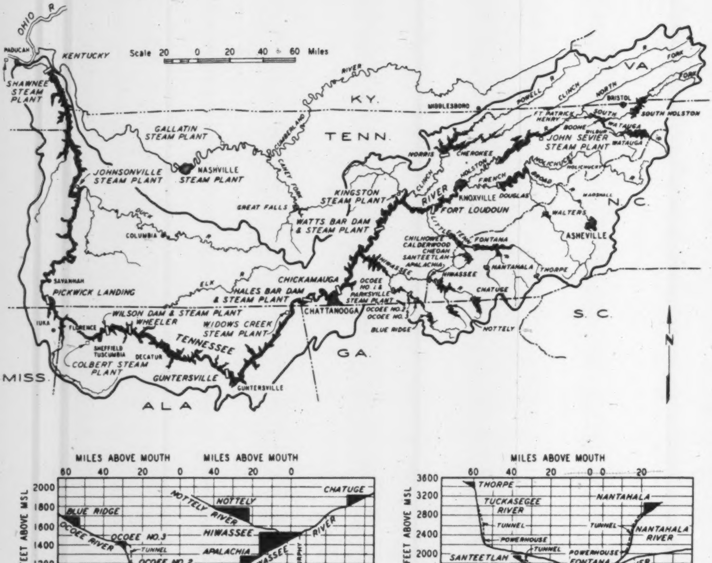
*Includes filter material


OHIO R
PADUCAH
KENTUCKY
Scale 20 0 20 40 60 Miles
SHAWNEE STEAM PLANT
KY.
TENN.
VA
N.C.
S.C.
GA.
ALA
MISS.
GALLATIN STEAM PLANT
NASHVILLE STEAM PLANT
JOHNSONVILLE STEAM PLANT
CUMBERLAND
RIVER
CANEY FORK
GREAT FALLS
KINGSTON STEAM PLANT
WATTS BAR DAM & STEAM PLANT
DUCK R
COLUMBIA
SAVANNAH
PICKWICK LANDING
ELK R
HALES BAR DAM & STEAM PLANT
CHICKAMAUGA
CHATTANOOGA
WILSON DAM & STEAM PLANT
WHEELER
WIDOWS CREEK STEAM PLANT
TENNESSEE
IUKA
FLORENCE
SHEFFIELD
TUSCUMBIA
DECATUR
COLBERT STEAM PLANT
GUNTERSVILLE
MIDDLESBORO
POWELL R
CLINCH
NORTH FORK
FORK
BRISTOL
SOUTH HOLSTON
FT PATRICK HENRY
SOUTH
BOONE
WILBUR
WATAUGA
JOHN SEVIER STEAM PLANT
NORRIS
CHEROKEE
HOLSTON
RIVER
FRENCH BROAD
NOLICHUCKY
KNOXVILLE
DOUGLAS
FORT LOUDOUN
MARSHALL
WALTERS
ASHEVILLE
LITTLE TENN
FONTANA
CHILHOWEE
CALDERWOOD
CHEOAH
SANTEETLAH
APALACHIA
HIWASSEE
NANTAHALA
THORPE
CHATUGE
OCOEE NO. 1 & PARKSVILLE STEAM PLANT
OCOEE NO. 2
OCOEE NO. 3
BLUE RIDGE
NOTTELY
N
MILES ABOVE MOUTH
ELEVATION - FEET ABOVE MSL
BLUE RIDGE
OCOEE RIVER
OCOEE NO. 3
TUNNEL
OCOEE NO. 2
POWERHOUSES
FLUME
OCOEE NO. 1
CHICKAMAUGA RES
NOTTELY
NOTTELY RIVER
HIWASSEE
APALACHIA
TUNNEL
POWERHOUSE
HIWASSEE RIVER
MURPHY
CHATUGE
HIWASSEE RIVER - MILES ABOVE MOUTH
THORPE
TUCKASEGEE RIVER
TUNNEL
POWERHOUSE
SANTEETLAH
CHEOAH RIVER
CHEOAH
CALDERWOOD
CHILHOWEE
FONTANA
LITTLE TENNESSEE RIVER
NANTAHALA
NANTAHALA RIVER
LITTLE TENNESSEE RIVER - MILES ABOVE MOUTH
WATTS BAR POOL
NORRIS
CLINCH RIVER
POWELL RIVER
CLINCH RIVER - MILES ABOVE MOUTH
PADUCAH
OHIO POOL NO. 52
KENTUCKY
PICKWICK LANDING
DAM NO. 1
WILSON
WHEELER
GUNTERSVILLE
HALES BAR
CHICKAMAUGA
CHATTANOOGA
WATTS BAR
FORT LOUDOUN
KNOXVILLE
DOUGLAS
CHEROKEE
TENNESSEE RIVER
HOLSTON R
LITTLE TENN R - MOUTH (See profile above)
CLINCH R - MOUTH (See profile above)
HIWASSEE R - MOUTH (See profile above)
FRENCH BROAD R
WALTERS
PIGEON R
NOLICHUCKY
BOONE
FORT PATRICK HENRY
SOUTH HOLSTON
WATAUGA R
SOUTH FORK
NORTH FORK
TENNESSEE RIVER - MILES ABOVE MOUTH

NOVEMBER 1956

FIG. A.—MAP AND PROFILE OF THE TENNESSEE RIVER SYSTEM

The Local Materials

The earth materials available where these three tributary stream dams are located were classified as follows: Nottely, sandy clay; Watauga, clayey sand and lean clay; South Holston, medium clay. At these locations there would have been a sufficient amount of these fine soils available for construction of an all earth dam, with rather flat face slope, but at each location there was also available a large quantity of rock for the construction of a rock fill dam. After due examination, the rock fill type with central earth core proved to be more economical. This type required less total yardage, construction time was reduced, and the diversion, power and outlet tunnels were considerably shorter. Rock classifications were as follows: Nottely, quartzite with 20 per cent mica schist; Watauga, quartzite; South Holston, sandstone with some shale.

Since the rock fills furnish most of the stability in this type of dam, wide variations of materials and dimensions of the core are possible to suit available local materials and relative costs of earth and rock. There will be a difference in settlement characteristics of the compacted earth core and the dumped rock fill, but with a heavy core this difference can have no injurious effect on the watertightness of the core.

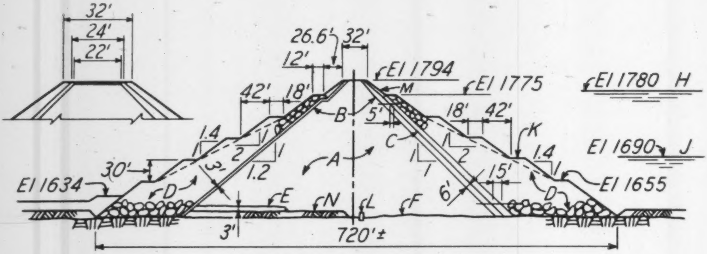
The heavy earth core can be made of almost any ordinary earth material. The governing criteria are permeability and compressibility. The material must be impervious when compacted and, therefore, must be capable of moisture control so that thorough compaction can be accomplished to provide the impermeability and to reduce the settlement in the finished dam.

Design of the Rock Fills

The rock fill must be made of sound rock having good compressive strength and durability against weathering. For rock fills the dry weights have been from 105 to 120 pounds per cubic foot. The quartzites, limestones, and dolomites of the region are generally acceptable, but the shales are generally unacceptable.

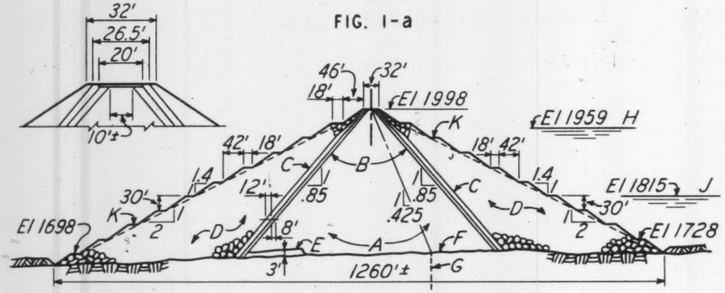
At South Holston Dam a high-grade sandstone was used and since the characteristics of this material were uncertain a large scale test was made to check the friction angle of dumped rock fill. Opening of the quarry at South Holston revealed that along with the sound sandstone were appreciable quantities of shale and of sandstone from which cementing material had disappeared. This uncemented sandstone had the appearance of being sound, but on handling it broke down into sand and fragments of stone. The apparatus shown in Fig. 2 was filled with 6- to 12-inch stone which was vibrated during placement. Concrete slabs were superimposed for added load, and the hydraulic jacks then applied horizontal force until actual lateral displacement of the frame and its contents took place. The tests showed an effective friction angle of 45 degrees for the sound sandstone.

When the stone was removed from the frame, several pieces were found to have actually broken, indicating the good interlocking action and crushing


32'
24'
22'
26.6'
32'
12'
El 1794
42'
18'
M
El 1775
El 1780 H
B
5'
18'
42'
K
1.4
1
2
1
1.2
A
C
1
1
2
El 1690 J
1.4
1
30'
3'
1.2
15'
El 1634
D
6'
D
El 1655
E
N
L
F
3'
720'±

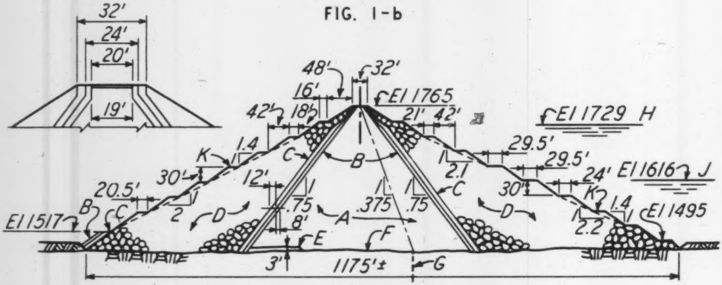
NOTTELY DAM

FIG. 1-a


32'
26.5'
20'
10'±
46'
32'
18'
El 1998
K
El 1959 H
42'
18'
C
B
18'
42'
1.4
1
.85
1
.85
C
1.4
1
El 1815 J
12'
30'
K
1
2
D
.425
D
1
2
30'
El 1698
8'
E
A
F
El 1728
3'
1260'±
G

WATAUGA DAM

FIG. 1-b


32'
24'
20'
19'
48'
32'
16'
El 1765
42'
18'
21'
42'
El 1729 H
1.4
K
C
B
1
2.1
29.5'
29.5'
1
30'
12'
1
24'
El 1616 J
20.5'
1
.75
.375
1
.75
C
30'
K
1.4
1
2
2.2
El 1517
B
C
D
A
D
El 1495
8'
E
F
3'
1175'±
G

SOUTH HOLSTON DAM

FIG. 1-c

A: Impervious rolled fill.-B: Fine filter.-C: Coarse filter.-D: Quarry run rock.-E: Drain blanket.-F: Weathered rock line.-G: Grout curtain.-El: Elevation in feet above mean sea level.-H: Maximum controlled level for multipurpose operation.-J: Minimum headwater.-K: Berms spaced 30 feet vertically.-L: Cut-off wall.-M: Riprap.-N: Overburden left in place.

FIG. 1.—SECTIONS OF THREE TVA ROCKFILL DAMS

resistance of the pieces. Tests of both the shale and uncemented sandstone showed a friction angle of about 30 degrees for each. The tests effectively verified the use of the generally assumed 45-degree friction angle of a good dumped rock fill.

Use of the weaker stone was allowed in the upstream rock fill if mixed with enough superior stone to have an average friction angle of more than 40 degrees. To compensate for this reduction in strength of the rock, the upstream surface slope was flattened to 1 on 2.2 in the lower part and 1 on 2.1 in the upper part. Only sound rock was used in the downstream rock fill.

Both upstream and downstream surface slopes at all three dams were established at 1 on 2 except the upstream slope at South Holston as mentioned above. At all dams the slopes were broken by berms spaced 30 feet vertically, the actual slope between berms being 1 on 1.4 with the average surface slope drawn through the centers of the berms. Rock dumped from trucks on a slope not more than 30 feet high does not roll away to any great extent and takes a natural slope of about 1 on 1.4. A 30-foot-deep layer of rock fill can thus be placed against the core along the full length of the dam with no subsequent handling except to clean up the berms for the sake of appearance.

Design of the Earth Core

For the three dams under discussion, the friction angles of the core materials have been from 23 to 30 degrees and the saturated weights from 120 to 130 pounds per cubic foot. In this type of dam cohesion of the core material does not have major influence on the strength of the maximum dam section. In the heavier Nottely core, cohesion of 500 pounds per square foot was included in the design. In the steeper cores of Watauga and South Holston cohesion was ignored. The soils used actually have a cohesion of 400 to 1000 pounds per square foot, adding to the computed safety factors.

As shown in Fig. 1 the core slopes at Nottely Dam are flatter than at Watauga and South Holston. This was due to the fact that core material at Nottely was easier to obtain than rock. Then, too, the dam was built on a crash schedule and it was known that considerable core could be placed before the quarry and crushing plant were ready to produce rock fill and filter material. This suggested that the core be as wide as practical consistent with safety. Generally the core precedes the rock fill and it should be capable of standing alone at some height above the rock fill. The circumstances called for flatter core slopes. (At Nottely the height of the earth core at one time led the rock fill by as much as 100 feet.) A slope of 1 on 1.2 was established for the downstream side of the core. For the drawdown condition on the upstream slope the rock fill is dry but the earth core is saturated, providing only buoyant weight for frictional resistance. So, to maintain the desired factor of safety, the upstream rock fill was made thicker within the established 1 on 2 outer slopes by making the upstream face of the core a 1 on 1 slope.

TVA has used the slip-circle method for the design of all fill dams. For the maximum dam section at Nottely the computed slip-circle factor of safety for the downstream slope with reservoir full and for the upstream slope with complete sudden drawdown was about 1.45. In the design the internal friction of the saturated earth fill was assumed to be 26 degrees. It was later found to be 30 degrees which increased the factor of safety.

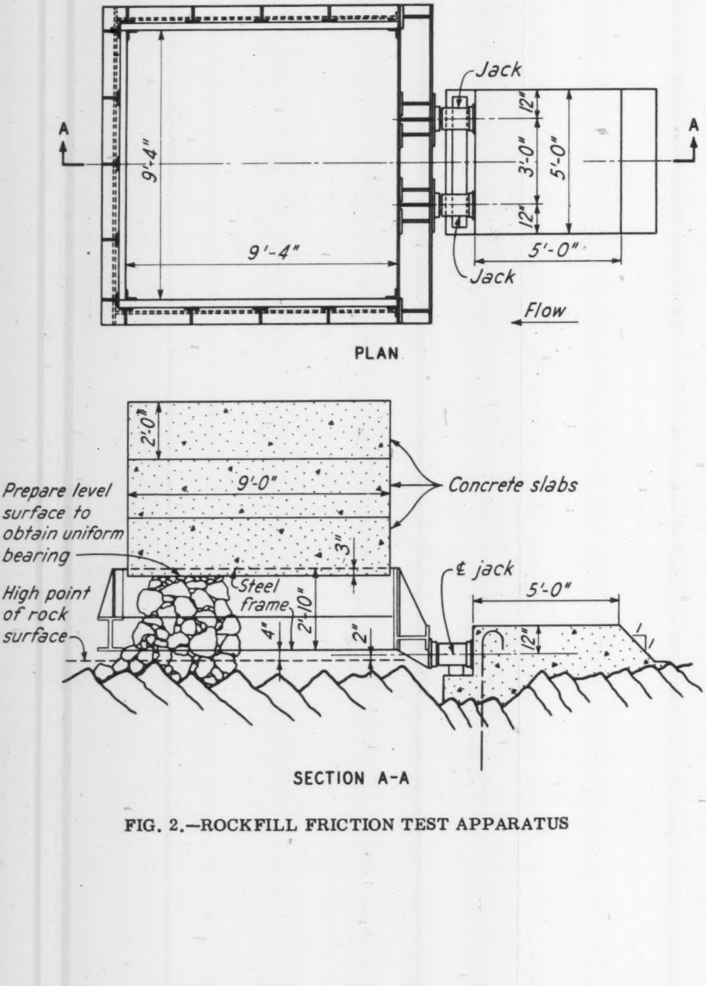

Jack
Jack
A
A
9'-4"
9'-4"
12"
3'-0"
12"
5'-0"
5'-0"
Flow
PLAN
2'-0"
9'-0"
Concrete slabs
Prepare level surface to obtain uniform bearing
3"
Steel frame
2'-10"
High point of rock surface
4"
2"
℄ jack
5'-0"
12"
SECTION A-A

FIG. 2.—ROCKFILL FRICTION TEST APPARATUS

At Watauga and South Holston, rock was abundant and readily accessible at both sites, so there was no expectation that the cores would seriously lead the rock fills. The core slopes could therefore be made relatively steep. At Watauga, core material was also abundant and economically obtainable, but at South Holston earth borrow involved a longer haul and more difficult excavation. The core slopes were set at 1 on 0.85 for Watauga and 1 on 0.75 for South Holston, both upstream and downstream, with the design rock slopes at 1 on 2 for both dams.

With this core shape it was assumed that the core would be fully saturated to full pool level, making the stability of the upstream slope for the complete drawdown condition identical with that of the downstream slope for reservoir full. The core material friction angles were 30 degrees at Watauga and 23 degrees at South Holston. The computed slip-circle safety factor for the maximum dam section was 1.7 for Watauga and 1.65 for South Holston.

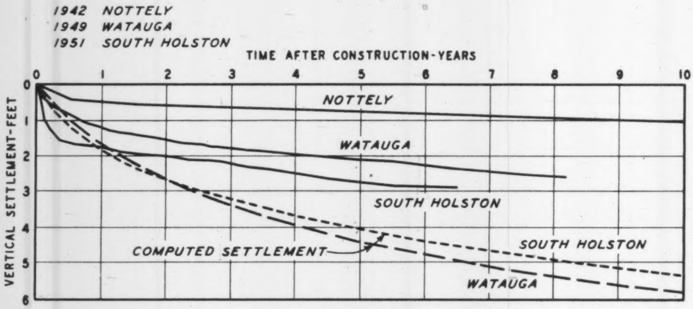
A foundation drain blanket was provided for the full length of Nottely Dam. It underlay somewhat more than the downstream quarter of the core. The design assumed that in this heavier core the saturation line would be lowered by the drain blanket. At Watauga and South Holston it was evident that drain blankets would be ineffective. The blankets shown on Fig. 1 for these dams cover only the exposed river bed where they might serve to intercept seepage through open rock seams and reduce uplift under the core.

Filter blankets were used between the earth and rock on both slopes to prevent the core material from being washed into the rock. Two zones were placed on all dams on the upstream side. Two similar zones were placed on the downstream side at Watauga and South Holston, but at Nottely there is only a single 3-foot-thick zone.

Construction Details

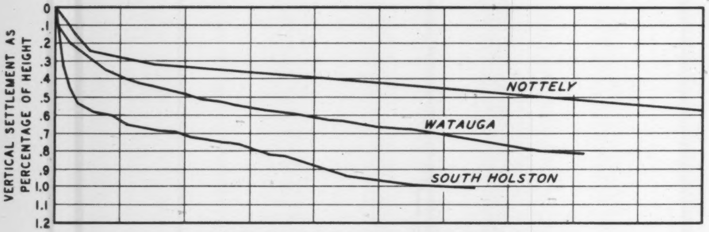
The treatment of the cutoff between dam and foundation varied to suit local conditions. At Nottely the overburden was thin except on the upper part of the left abutment. On the lower left abutment, across the channel, and on the entire right abutment all overburden was removed from the area to be covered by upstream and downstream rock fills and by the upstream half of the core. A trench was excavated to impervious rock just upstream of the center line of dam. A concrete cutoff wall in the trench was keyed 2 feet into rock and projected 4 feet up into the core. No grout curtain was required in this rock foundation. Good overburden was left in place under the downstream half of the core. On the upper left abutment, where the height of dam became less than 70 feet, two sections were used. Where the dam was 30 to 70 feet high, overburden was removed for the entire base of rock fills. Overburden was left in place under the entire core, the cutoff wall was omitted, and the upstream face of the overburden left in place was blanketed with 15 feet of rolled core material. Where the dam was less than 30 feet high, core and rock fills were founded on stripped overburden.

At both Watauga and South Holston, overburden was thin and was removed under the entire dam area except in a few areas where good overburden was left in place under the downstream half of the core. A grout curtain was provided in the rock foundation at the upstream quarter point of the core. The traditional cutoff wall was omitted. Its only function could be to make a firm connection between foundation and core in one isolated spot and increase the seepage path along the foundation plane by the total of its peripheral dimensions. With good compaction of the core on the foundation a successful



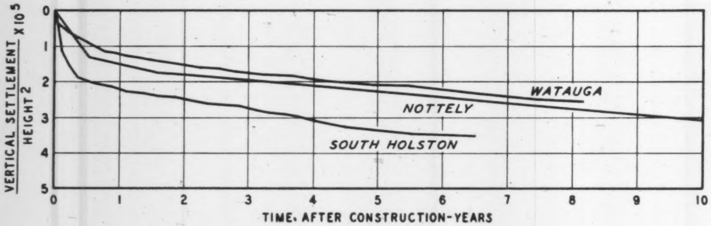
SETTLEMENT

FIG. 3-a



SETTLEMENT AS FUNCTION OF HEIGHT

FIG. 3-b



SETTLEMENT AS FUNCTION OF SQUARE OF HEIGHT

FIG. 3-c

FIG. 3.—TIME-SETTLEMENT CURVES

cutoff is accomplished. The presence of a cutoff wall projecting into the fill makes equipment compaction of fill near the wall difficult and calls for inefficient hand tamping. A trench 6 feet wide was excavated 2 feet deep into the rock on the line of the grout curtain and filled with concrete. This concrete cap sealed the top of the grout curtain and provided a working surface for drilling and grouting operations.

Characteristics of the foundation and fill materials were determined from preliminary investigations and conservative values of these characteristics were used in the design. Control of construction procedures to guarantee the design strength was maintained by the field engineers. All earth fill was compacted by spreading in thin layers and rolling with sheepsfoot rollers. Any overburden left in place under earth fills was well harrowed and recompacted with sheepsfoot rollers the same as the fill itself to blend fill with foundation and obliterate the junction.

Rock fills were placed by dumping from trucks. At each point of dumping one or more streams of water at 100-psi pressure from a 2-inch nozzle removed small fragments from the contact between the larger rocks and accelerated the settlement and adjustment of the fill.

Significant settlements were expected in these high dams, and so the tops of the dams were cambered to extra height to maintain future freeboard. Possible settlement was estimated from laboratory tests of the earth materials during fill construction. At South Holston, for example, the ultimate settlement was estimated to be about 18 feet and therefore the maximum section of the dam was raised 10 feet, corresponding to about the 60-year estimated settlement. Actual measured settlement at both Watauga and South Holston has been substantially less than the estimated. To avoid a sagging appearance near the top of the dam, the three upper rock berms at Watauga and South Holston were cambered successively greater amounts to build up the top camber.

Measured Settlement of Core

Continuous records of settlement have been kept on Nottely, Watauga and South Holston Dams. Observations at Watauga and South Holston started one month, and at Nottely four months, after completion of the fill and dam closures. The reservoirs were nearly full in eight to nine months, and since initial filling they have experienced seasonal drawdowns and refillings during flood control and power storage operations.

At Nottely the fill was placed in one period; at Watauga in two periods; and at South Holston in three periods. The work periods were separated by the typical winter rainy season.

All the observation points are located on the central earth core on the maximum section and represent consolidation of the earth core under its own weight and weight of the superimposed rock shoulder. No settlement measurements were made on the rock fill itself.

Time-settlement curves for the cores of these three dams are shown in Fig. 3, plotted as direct settlement, settlement as a percentage of height, and settlement as a function of the square of the height. After ten years the top of Nottely Dam had settled 1.1 feet or 0.6 per cent of the 184-foot height. Watauga Dam in eight years settled 2.6 feet, or 0.8 per cent of its 318-foot height, and South Holston's 285-foot height was decreased in 6-1/2 years by 2.85 feet or 1.0 per cent. At age six and one-half years (South Holston's age),

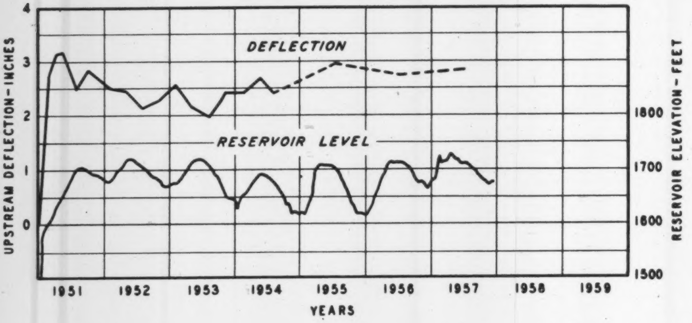


FIG. 4.—DEFLECTION CURVE FOR SOUTH HOLSTON DAM

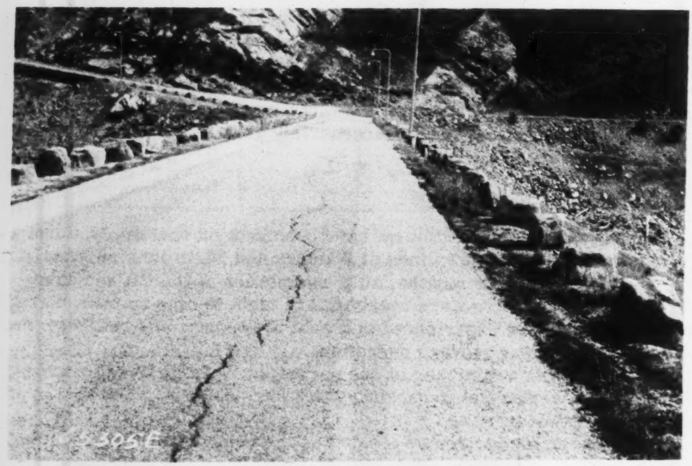


FIG. 5.—CRACKS FORMING ALONG UPSTREAM SHOULDER OF CORE OF WATAUGA DAM

the percentage settlements of Nottely and Watauga were 0.46 and 0.74 compared with South Holston's 1.0 per cent.

Since consolidation of a specimen varies directly with its length and roughly with the pressure applied to it, the settlement of fills would be expected to vary approximately as the square of the height of the fills, providing the fills have the same shape and are made of materials with similar consolidation characteristics. The chart of this function shows the relative settlement curves proportionately closer together than either the direct settlement or the percentage settlement, the Nottely and Watauga curves practically coinciding.

Theoretically, the rate of settlement at South Holston and Watauga should have been about the same. At Watauga the dam is slightly higher (320 feet compared with 290 feet), producing higher compression. The rate of placing the fill was much faster at Watauga (13 months) than at South Holston (24 months). Therefore, the consolidation obtained during construction is greater at South Holston. In the detailed computation of consolidation these conflicting influences produced closely similar time-settlement curves for both dams for the first three years after completion of the fills. Thereafter the computed curve for the higher Watauga Dam showed a little greater settlement.

After the first year the curves of actual settlement of the two dams are nearly parallel, both at a slower rate than the computed. The only significant difference occurred in the first year, when South Holston experienced settlement at a much faster rate than expected. Here the top ten feet of the nominal height of the core and the superimposed 10-foot camber were compacted by truck and tractor rather than by sheepsfoot rollers. It is suspected that this contributed materially to the rapid early settlement. Fig. 3-a shows comparative computed and actual settlement curves.

The vertical settlement curves for Watauga and South Holston seem to show some response to reservoir fluctuation, as evidenced by the local steepening of the curve at the end of each of the earlier years when readings were taken with sufficient frequency to disclose the effect. (Figs. 3-a and 3-b.)

Cracks in Top of Dam

Unequal settlement of the central core and the rock fills has resulted in considerable cracking, with the resultant maintenance expense, along the shoulders of the core at all three dams although it is much less at South Holston than at Watauga and Nottely. In each case the rock fills settle more than the cores.

Progressive settlement along both shoulders for the full length of the roadway at Watauga has been noticeable for several years. Cracking on each side of the pavement (see Figs. 5 and 6) has been filled with asphalt several times. These cracks seem to follow roughly the edges of the earth core beneath the pavement. In 1957 there was a vertical displacement of up to four inches in a 50-foot section on the downstream side near the right end. This larger crack was enlarged and backfilled with clay and surfaced with hot mixed asphaltic concrete. A shorter area on the left end on the upstream side required similar treatment. The greatest settlement appears to have occurred at the extreme left end on the upstream side at the edge of the shoulder.

In contrast, at South Holston the roadway is cracked at only one short section on the downstream side at the left end where a roadway fill crosses the core to join the top of the dam. Widening the top of the core, which was

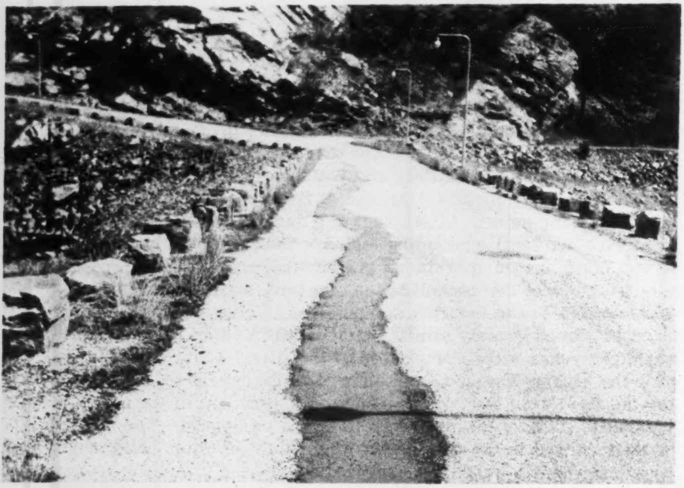


FIG. 6.—WATAUGA DAM, CRACK ALONG DOWNSTREAM SHOULDER OF CORE CUT OUT, FILLED AND SEALED

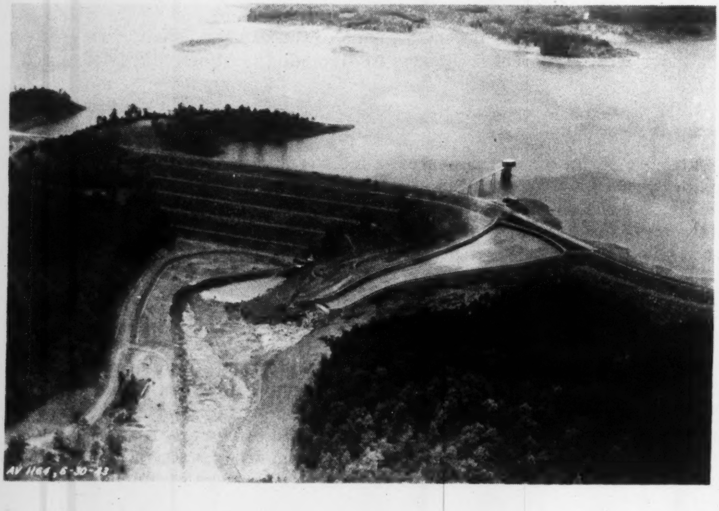


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FIG. 7

done for construction convenience, was beneficial. A comparison of the top details of the cores at Watauga and South Holston is shown in Fig. 1. There is little, if any, change in the relative elevation of the top of the roadway and the point of the shoulders at South Holston; whereas, at Watauga the shoulder in some spots is as much as six to eight inches below the center of the road.

Severe similar cracking at Nottely was observed in 1955, although prior to that time upstream settlement of the rock fill and resultant repairs had completely buried the curb under 6 to 10 inches of bituminous material. The downstream curb still retained its original appearance. In 1956 a small crack opened up along the downstream shoulder, but it was filled and today both sides seem to be stationary.

The 1955 crack extended for about 600 feet along the high part of the dam. It was in the pavement and generally was located 5 to 6 feet from the upstream edge of the embankment, but in some places it was as much as 10 to 12 feet. The crack was generally 1/4 inch or less wide, but in one or two places and over short distances the width was as much as 1-1/2 inches. Its depth exceeded 6 feet as determined by probing rods and 4-inch auger holes, the latter being stopped by rocks. There was no significant indication of any differential settlement between the edges of the crack. The pavement sloped downward between the crack and the upstream edge of the embankment.

Many ideas, some unique and possibly far-fetched, have been presented to explain the cause or causes of the cracks at these three dams. The seemingly apparent cause was expressed above—unequal settlement of the core and rock, which is influenced by differences in materials, design, and construction methods.

It is difficult to interpret the differential settlement of core and rock fill although it is generally recognized that some rapid adjustment and settlement of the rock fill is to be expected. The compaction of core material is pretty well known and settlement can be anticipated from laboratory tests. Rock fills, however, due to varying qualities of gradation, hardness, durability, roughness, and construction methods cannot be so anticipated.

As evidenced by the settlement curves (Fig. 3) good compaction was achieved in all cores. Settlement after 6-1/2 years to 10 years did not exceed 1 per cent of the height. It is believed that the low 0.6 per cent settlement at Nottely was due to the sandy soil which had a near optimum water content. At Watauga and South Holston moisture control was troublesome with the soils in the borrow pits being generally on the wet side of optimum. Aeration and drying of the soil before placing consumed much valuable time and compaction may have been hurried.

The cracking described does not endanger the stability of the dams as the friction in the rock fill and cohesion and friction in the uncracked portion of the impervious core provide adequate safety.

Deflection

Nottely Dam showed appreciable lateral deflection during reservoir filling. Upon first filling, the top of the dam at the maximum section deflected downstream nearly three inches. The deflection decreased one inch during the next drawdown and increased one-half inch on refilling.

Watauga Dam showed no significant deflection from reservoir filling. No readings have been taken at Nottely and Watauga for a number of years.

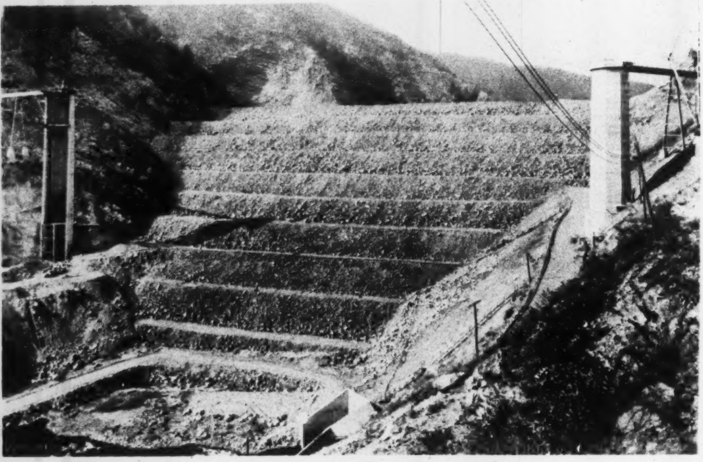

FIG. 8.—UPSTREAM VIEW NEAR END OF CONSTRUCTION OF WATAUGA DAM

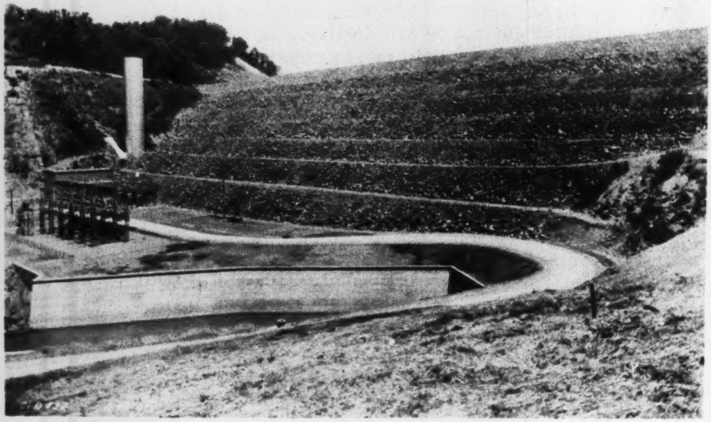

FIG. 9.—DOWNSTREAM VIEW OF SOUTH HOLSTON DAM

South Holston Dam has shown unusual lateral deflection of the top of the dam. A point near the center of the dam deflected upstream more than three inches during the initial loading of the dam in 1951 by the reservoir water. This deflection has varied during the subsequent seven years as shown by the curves on Fig. 4. A response of about one-half inch to reservoir changes in the early years is indicated. The upstream deflection might have been due to tilting of the dam foundation during reservoir loading. Substantial width of the valley floor at the dam site and relatively weak shale and sandstone foundation rock might tend to produce such a tilting effect. However, there is no survey information to substantiate this theory.

Seepage and Toe Drainage

Little information is available concerning the seepage through the core and toe drainage below the three dams herein described. It must suffice to say that toe drainage has been low. At Nottely an attempt has been made to catch the drainage from the right bank and measure its flow over a weir. Hydrographs of flow compared with rainfall at Nottely during 1954 indicated that the drainage was about 6 gpm during dry periods and about 30 gpm after rainy periods.

The collection of accurate seepage and drainage flows is extremely difficult and unreliable. Rainfall, temperature, evaporation, and geology all tend to discourage the observer from placing too much reliance on his records.

Since visual inspection has consistently shown low flows, it seems not too important to worry about the actual amount of seepage.

CONCLUSIONS

In TVA dams up to 300 feet high the settlement of the core in six to eight years has been less than predicted from computation of simple vertical consolidation, except for vagaries in the first year or so.

The rock fill shoulders have generally experienced more settlement than the cores during the early years. Efforts should be directed toward reducing this difference in settlement as much as possible, and toward minimizing the effect of the difference. The rock in the fill should be sound and durable and it should be placed under specifications which accomplish its adjustment to stable position. If the dam is to have a paved roadway, the core at the top should be the full width of the roadway including curbs and continuous type guard rail. A stone guard fence, well adapted to these dams, was used at Watauga and South Holston. Located on the rock fill shoulder, it was made of selected large stones spaced about seven feet on centers.

Little can be deduced from the deflection experience of these dams except that they show a certain elasticity of deflection under varying reservoir load.

The spectacular in dam performance is usually associated with inadequacy or failure, of the whole or of a part. The performance of TVA's heavy central core rock fill dams has been entirely adequate, therefore not spectacular.

DISCUSSION

F. L. LAWTON,[3] F. ASCE.—The authors' treatment of the design and construction features of the Nottely, Watauga and South Holston dams is valuable. It is greatly enhanced by the discussion of operating experience.

The advantages of the central-core rockfill dams are well stated. The authors might have added that these probably apply equally well to rockfill dams with sloping impervious core. The type is essentially dictated by the relative availability of suitable material for the core. Both types appear much more resistant, inherently, to earthquake damage than any type of concrete dam.

Reference is made to the types of rock used for fill at the three dams, i.e. quartzite with 20 per cent mica schist at Nottely, quartzite at Watauga, and sandstone with some shale at South Holston. It would have added to the value of the paper if the authors had given a brief account of the principal geological features in the cross sections at the sites. Information as to strike and dip of the formations would also have been useful.

The unequal settlement of the central core and rock shoulders, which has been noted at the three dams, particularly at Watauga and Nottely, gives rise to the question: Do the rock shoulders in reality slide, to a degree, along the contact zones? This phenomenon appears to be a weakness of the central-core rockfill dam as compared with the sloping-core type.

The data reported for the lateral deflection is interesting. The rebound at Nottely dam and the upstream deflection for South Holston dam are noteworthy and confirm observations elsewhere. The implication, for South Holston, "The upstream deflection might have been due to tilting of the dam foundation during reservoir loading" is an interesting one and might be the explanation of what is, otherwise, difficult to explain.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3069

ROCKFILL DAMS:
DAMS WITH SLOPING EARTH CORES

By James P. Growdon,[1] F. ASCE

With Discussion by Messrs. Torald Mundal; John Lowe, III; and James P. Growdon

SYNOPSIS

This paper briefly reviews the principles which govern the design of a rockfill dam and discusses six rockfill dams with which the author has been intimately acquainted since completion of the Nantahala Dam. Bear Creek Dam, one of these six, incorporates most improvements in design and construction learned from experience.

INTRODUCTION

The conservation and control of water is vital in many parts of the world and is becoming increasingly important in all places where resources are being developed and population increased. Most projects for the control and use of water include a dam at some place in their progress schedule. During the last 30 years a great many dams of widely varying types have been constructed. Particularly in the United States, most of the better dam sites have been occupied leaving the less desirable sites for present and future development. The high cost of building dams has resulted in a search for better and more economical types. As a result of this search the rockfill dam with sloping impervious earth core was developed. The first dam of this type to be constructed was described elsewhere.[2]

The economy and performance of the Nantahala Dam has been so satisfactory that several similar dams have been built and others are being designed and constructed. This paper is limited to a brief review of the principles which govern the design of a rock fill dam and a discussion of the six rock fill dams with which the writer has been intimately acquainted since the

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Note.—Published essentially as given here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1743. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[2] "Rockfill Dams: Natahala Sloping Core Dam," by James P. Growdon, Transactions, ASCE, Vol. 125, Part II, 1960.

completion of the Nantahala Dam. The major part of the discussion will concern the Bear Creek Dam which was completed in 1953 since it incorporates most of the improvements in the design and the construction of rock fill dams which have been learned from experience.

The rock fill dam should be constructed only on a suitable site. The first requirement is a supply of sound rock adequate for the structure. Sound rock may be described as rock which will not disintegrate in the quarry nor in the handling; which is strong enough to sustain the weight of the dam and the water load and which will not disintegrate rapidly when exposed to weather. The second requirement is an adequate supply of earth suitable for the impervious core. Characteristics of "suitable earth" may vary widely without effecting the usability for an impervious core provided its characteristics are taken into account in the design. If the site permits the use of a diversion tunnel as a pressure conduit after the project has been completed and if the quarry can provide an adequate spillway the economy of the project will be enhanced.

Design

The design of a rock fill dam with sloping earth core is quite simple. It consists of the basic triangular rock section dumped at the natural angle of repose, with an impervious earth core on the upstream slope from which its separated by a suitable filter to prevent the migration of the smaller particles into the voids below. The upstream surface of the earth core is protected by a filter and a layer of rock thick enough to protect the core from wave action and to prevent it from sloughing under sudden draw down.

With one exception dams of this type so far constructed have had rock foundation from which the overburden has been removed. Any foundation which can be described as rock or shale is entirely adequate for even a high rock fill dam. Alluvial material filling a river channel also provides an adequate foundation. The foundation problem is greatly simplified because there is no sharply differential loading on the foundation. Water load is transmitted through the rock fill to the foundation normal to the earth core, so that the heaviest water load reaches the foundation through the shortest path.

A seal trench should be excavated to groutable rock where the earth core is in contact with the foundation. This seal trench should be grouted to whatever degree is necessary to limit the passage of water through the foundation to an acceptable amount. The core is placed directly on the rock in the seal trench.

Characteristics of the earth core can readily be determined in a soils mechanic laboratory and the impervious core designed to suit the characteristics of the material available. The thickness of the core should be such as to limit the passage of water through the dam to an acceptable amount. It should be wide enough to permit placing with mechanical equipment and thick enough to prevent rupture due to differential settlement of the basic rock fill. The lower and upper filter zones can be made of sand and gravel or crushed rock screened and sized so that "The 15% size of the filter shall be larger than 5 times the 15% size and smaller than 5 times the 85% size of the adjacent filter zone. The 15% size and the 85% size as used in this specification are sieve sizes which pass 15% and 85%, respectively, of the material". The upstream rock blanket is designed to prevent the sloughing of the core under conditions of a sudden draw down. The water impounded by the dam compresses the earth core against the filter material below it so that a core of moderate thickness contains very little water and its angle of internal friction is much greater than it would be for the same material in saturated condition.

All rock dams settle due to the break down of the point contact between the rocks comprising the basic rock fill and because fine material which may separate, some of the larger rocks will in time migrate into the voids below. Settlement is both downward and downstream. The downstream settlement being generally 1/2 magnitude of the vertical. The basic rock fill should be designed convex upstream so that when settlement occurs it will further compact the earth core. The dam should be given a camber so that the center of the structure will be at least as high as the abutments after a long period of settlement.

Existing Dams

Fig. 1 shows the profile of the existing rock fill dams referred to herein. On this you can see that the dams vary in height from 76 feet to 317 feet. With one exception the downstream slope is the natural angle of repose which is substantially one vertical on 1.3 horizontal. The downstream slope of the Kenney Dam consists of a series of berms which brings the average slope to one vertical on 1.75 horizontal. In each of the dams shown the upstream cofferdam has been incorporated in the structure. The upstream slope of the 7 days shown varies from one vertical on 2 horizontal to one vertical on 2.5 horizontal with a steeper slope at the top of all except Kenney Dam.

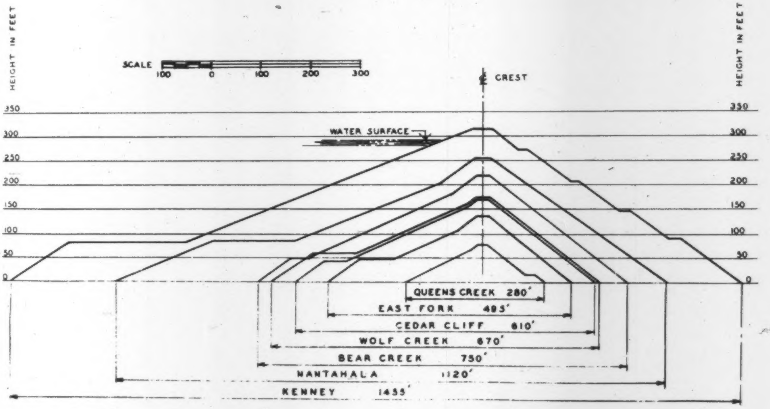

FIG. 1

Figs. 2 through 21 show several of the dams under construction and completed.

The rock used in Nantahala was very hard massive arkose. Quarrying methods produced rock of all sizes. Some single rocks were a full load for a truck. The rock at Queens Creek was also arkose but much less massive. Rock at Cedar Cliff, Bear Creek, Wolf Creek, and East Fork is a good quality shist much softer than the arkose. The rock used for the Kenney Dam was very sound hard basalt.

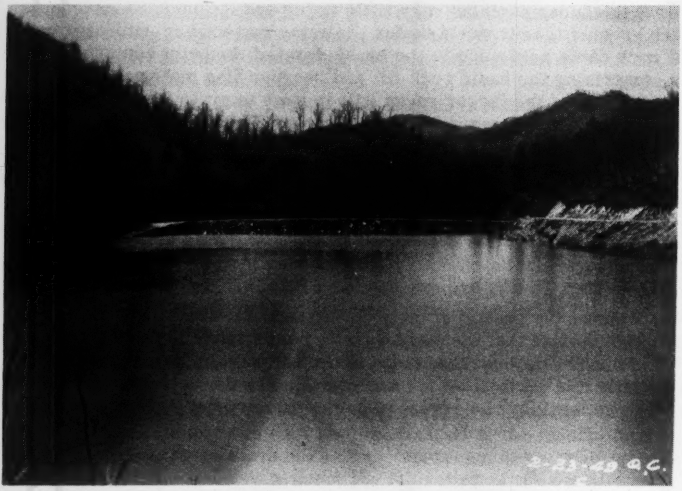

FIG. 2.—QUEENS CREEK DAM, RESERVOIR FILLED

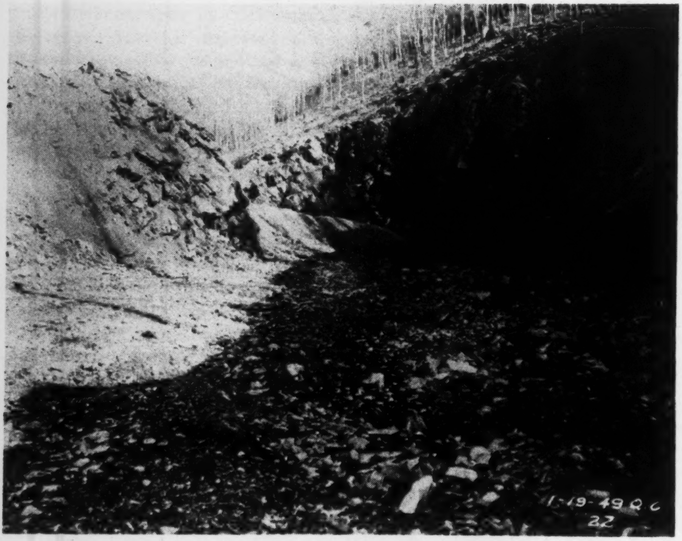

FIG. 3.—QUEENS CREEK, SPILLWAY CHANNEL

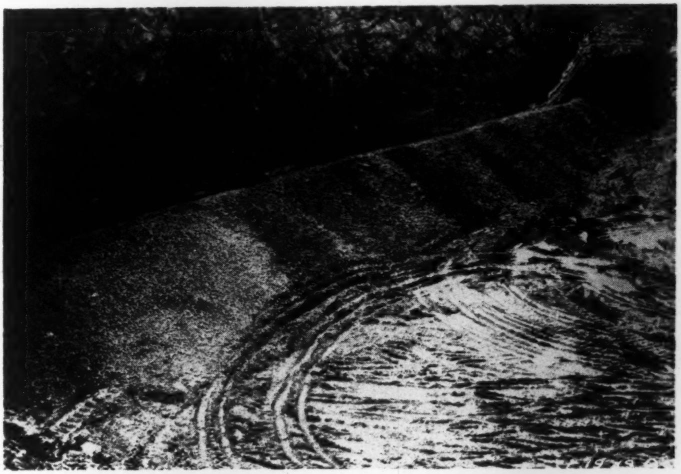

FIG. 4.—QUEENS CREEK, FUSE PLUG CONTROLLING SPILLWAY

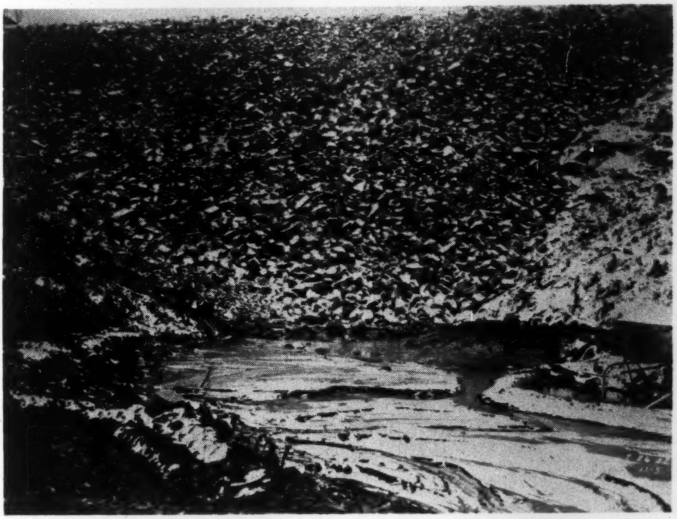

FIG. 5.—NANTAHALA DAM, DOWNSTREAM FACE

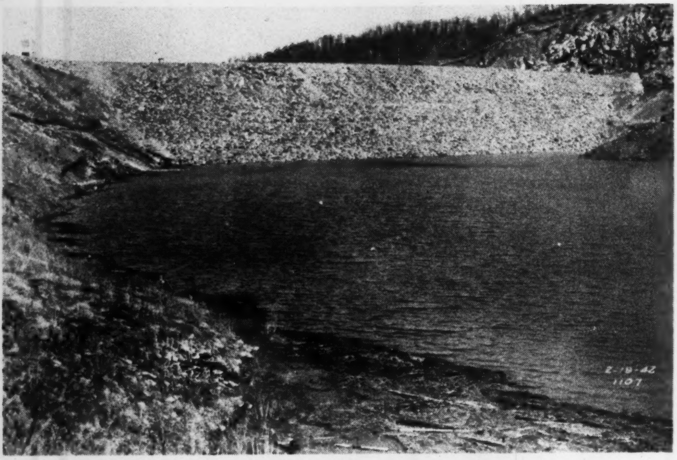

FIG. 6.—NANTAHALA DAM, UPSTREAM FACE WITH PARTIALLY FILLED DAM

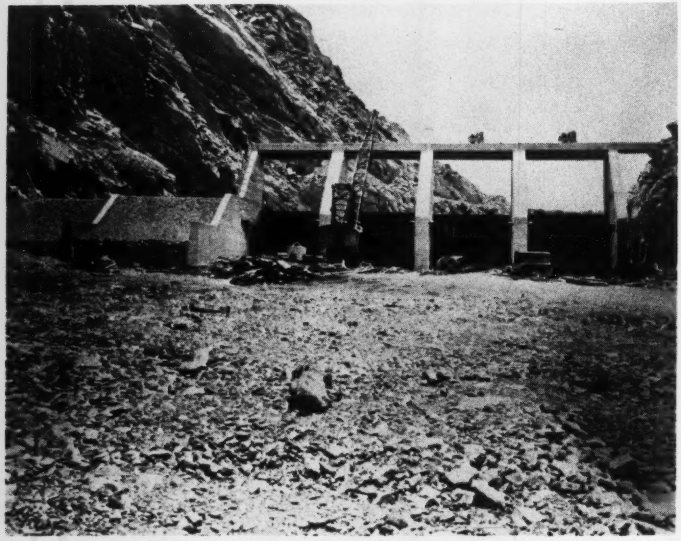

FIG. 7.—NANTAHALA, FLOOD GATES AND FUSE PLUG

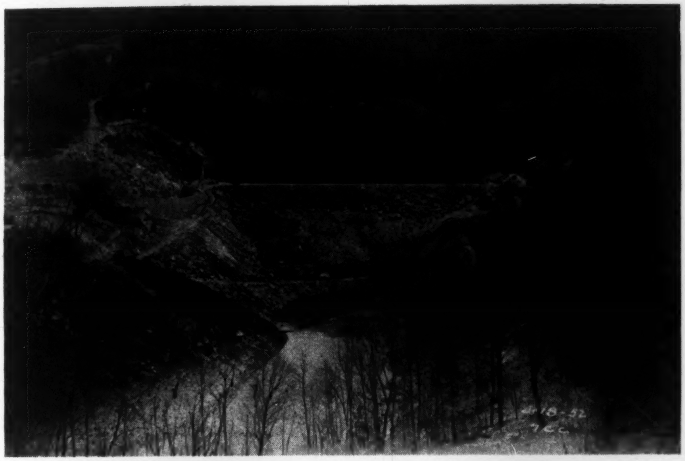

FIG. 8.—CEDAR CLIFF DAM, UNDER CONSTRUCTION

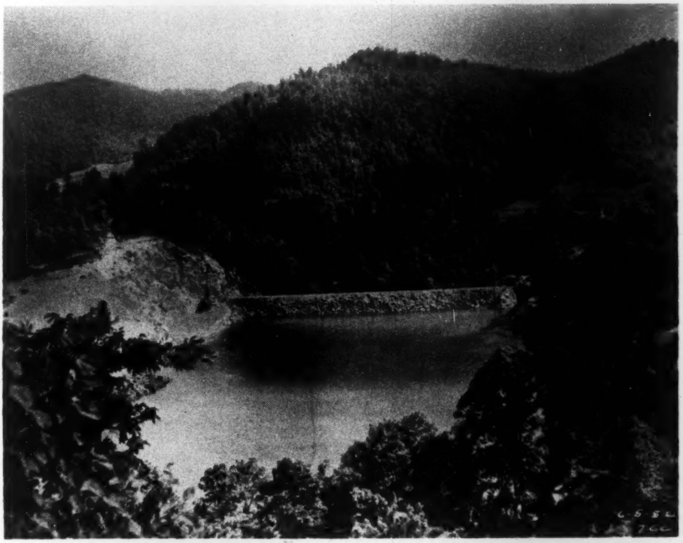

FIG. 9.—CEDAR CLIFF DAM, PARTIALLY FILLED RESERVOIR

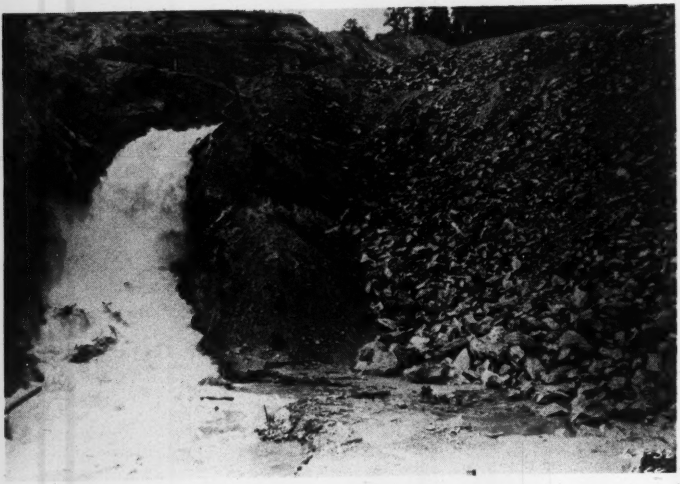

FIG. 10.—CEDAR CLIFF DAM, CASCADE SPILLWAY DISCHARGING 3,000 cfs

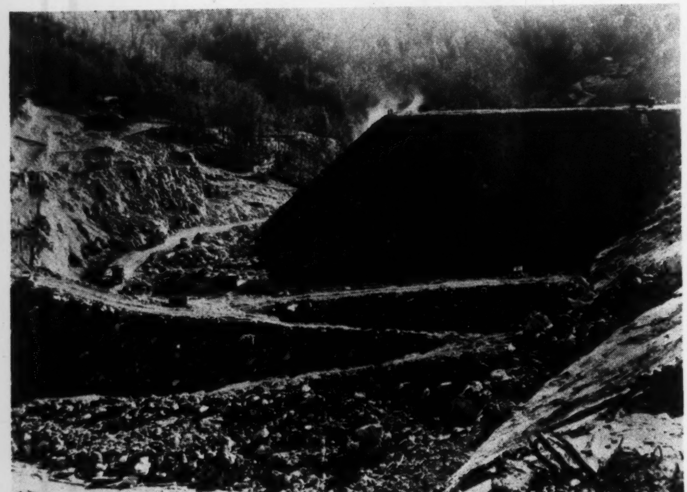

FIG. 11.—BEAR CREEK DAM, UNDER CONSTRUCTION

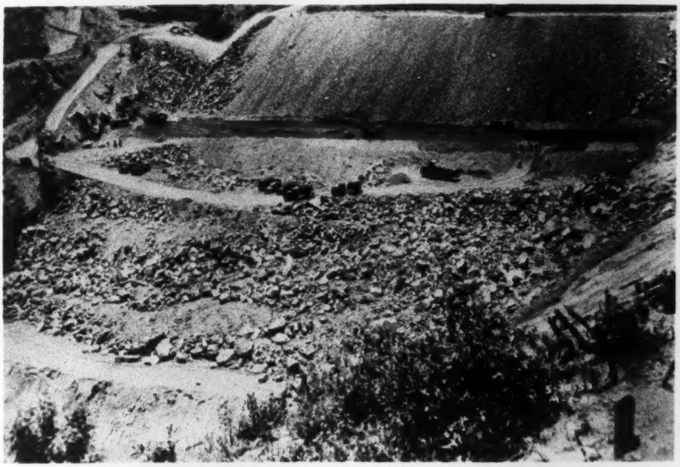

FIG. 12.—BEAR CREEK DAM, UNDER CONSTRUCTION

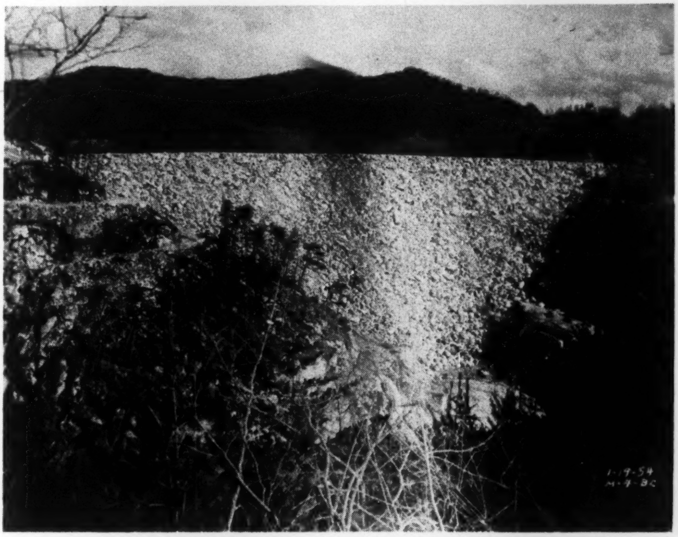

FIG. 13.—BEAR CREEK DAM, DOWNSTREAM FACE

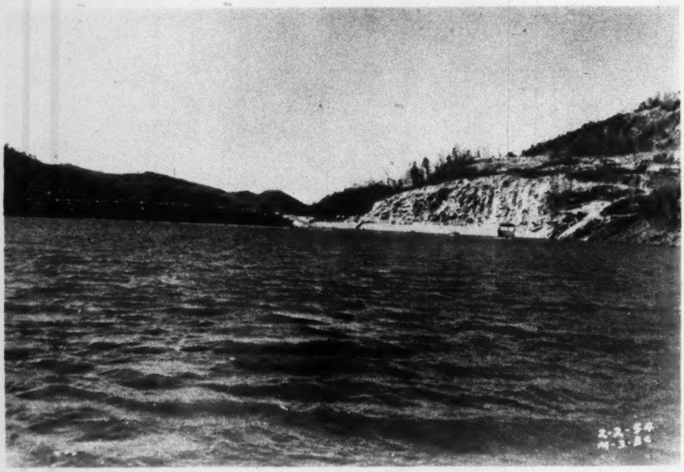

FIG. 14.—BEAR CREEK DAM, FULL RESERVOIR

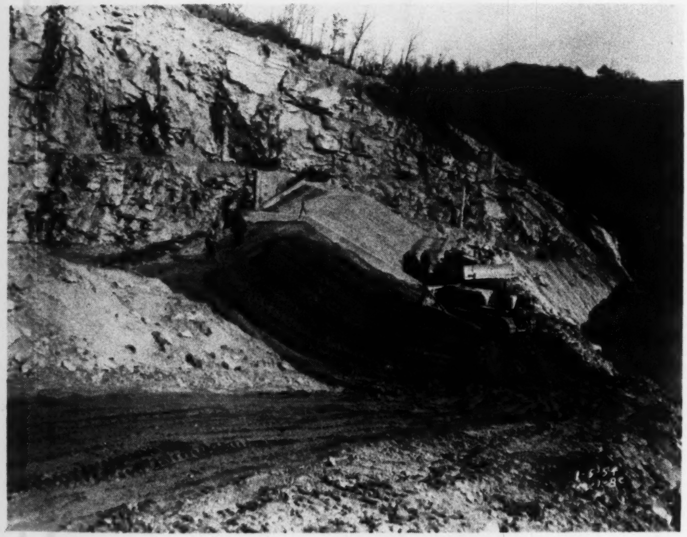

FIG. 15.—BEAR CREEK DAM, CONSTRUCTING FUSE PLUG

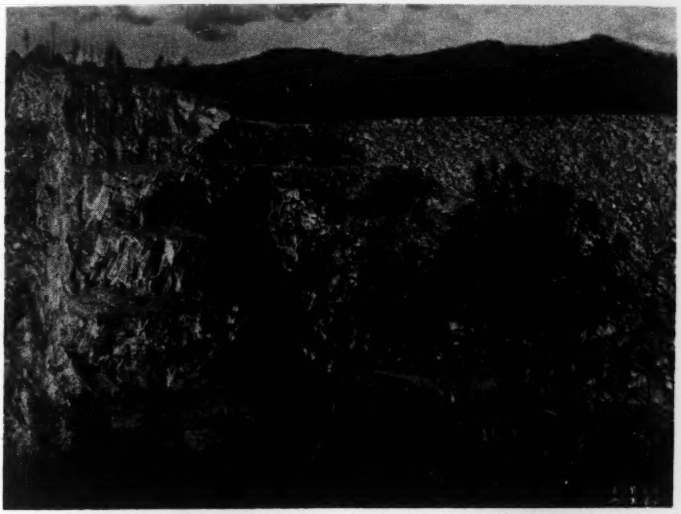

FIG. 16.—BEAR CREEK DAM, CASCADE SPILLWAY

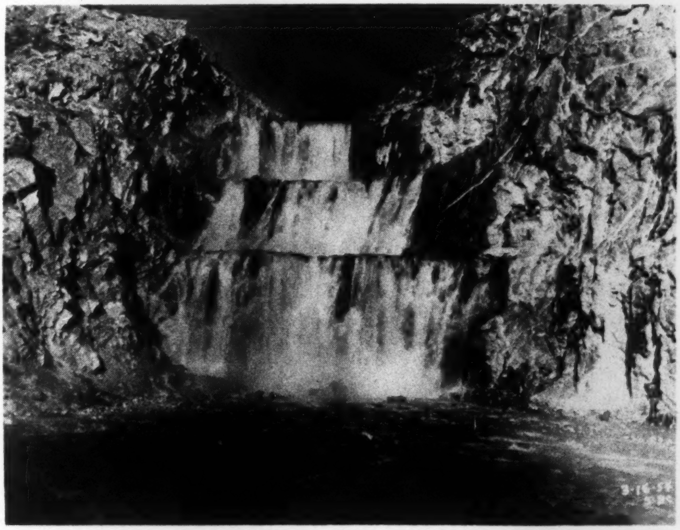

FIG. 17.—BEAR CREEK DAM, CASCADE SPILLWAY DISCHARGING 2,000 cfs

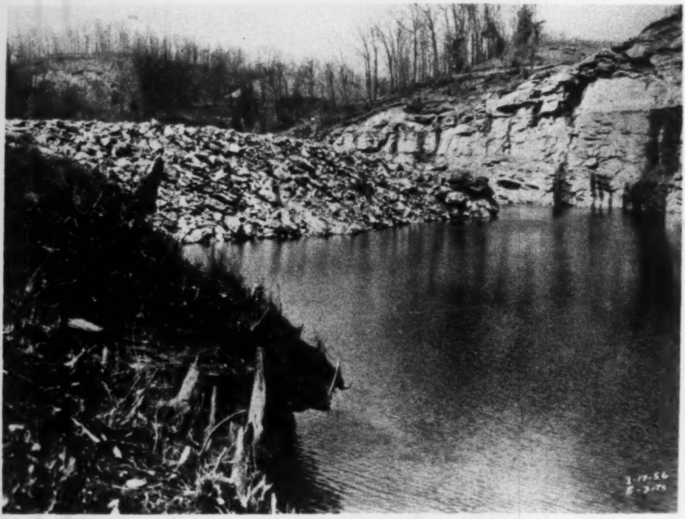

FIG. 18.—TENNESSEE CREEK DAM, PARTIALLY FILLED RESERVOIR

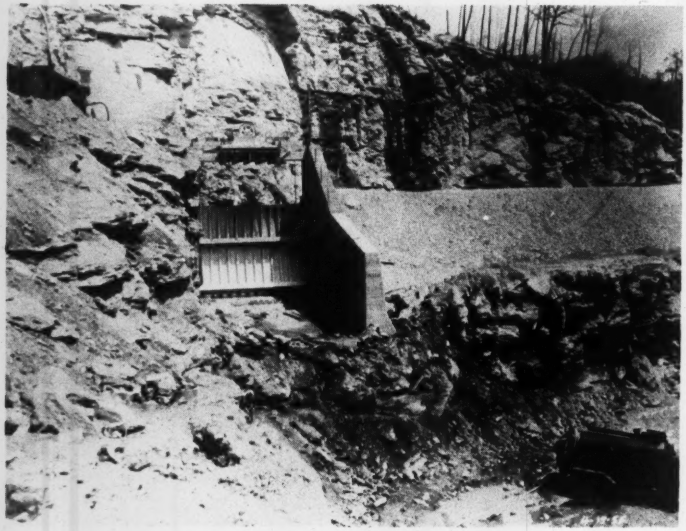

FIG. 19.—TENNESSEE CREEK DAM, TAINTER GATE AND FUSE PLUG CONTROLLING SPILLWAY

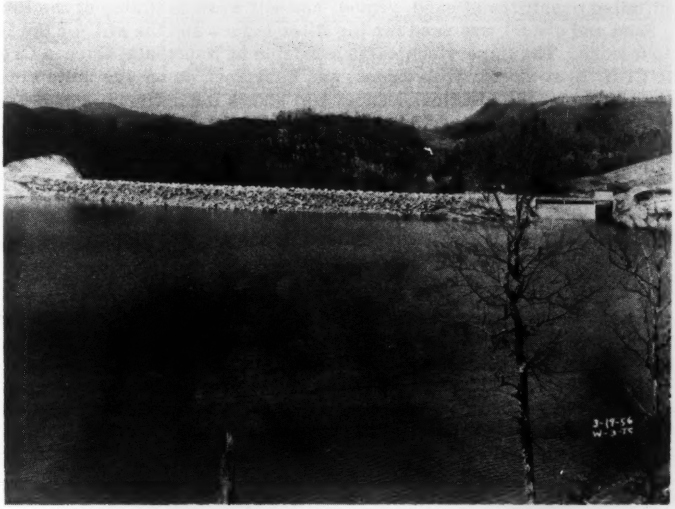

FIG. 20.—WOLF CREEK DAM, PARTIALLY FILLED RESERVOIR

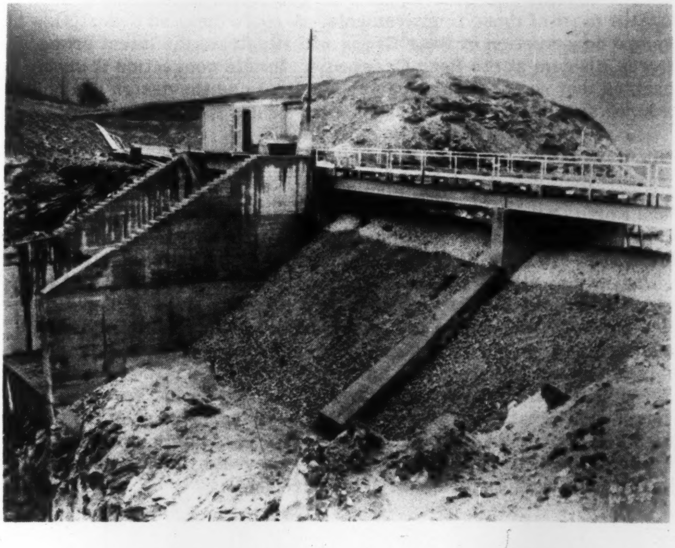

FIG. 21.—WOLF CREEK DAM, TAINTER GATE AND FUSE PLUG

Unlimited quantities of sand, gravel, and silt were available at the Kenney site. Sand and gravel was used for the filter layers and the silt for the impervious core. The impervious earth available at Nantahala, Queens Creek, Cedar Cliff, Bear Creek, Wolf Creek, and East Fork dams was quite similar although by no means identical. Table 1(a) shows the optimum soil conditions, Table 1(b) shows the in place soil conditions, and Table 1(c) shows the general soil characteristics for each of these six dams. Table 1(d) shows the vertical and horizontal settlement and the leakage.

You will note that the settlement varies from 2.85 feet vertical, 1.53 feet horizontal to 0.28 feet vertical and 0.18 feet horizontal at Queens Creek. The amount of settlement is partly determined by the construction procedure.

Leakage

There has been no measurable leakage passed the Kenney Dam. In case of the other dams leakage has been collected immediately below the dam and measured when the reservoir was full during dry weather. The leakage varies from 14.6 cubic feet per minute at Wolf Creek to 2.67 cubic feet per minute at East Fork. It is probable that most or all of the measured leakage is through the foundation rather than through the impervious core. The small amount of water which passes through the core would certainly be evaporated before reaching the point where it can be measured.

Bear Creek Dam

Experience gained in constructing these several rock rill dams has resulted in improving some features of the design and the construction procedures used at Nantahala. The Bear Creek Dam completed in November 1953 incorporates most of these improvements. A more detailed description of the design and construction of Bear Creek will illustrate the latest procedures for a rock fill dam at the Bear Creek site. In this connection it should be emphasized that no two dam sites are identical and the design and construction procedures should be varied to suit the particular site.

The Site

The Bear Creek site is on the East Fork of the Tuckasegee River in western North Carolina. The drainage area is 75.3 square miles. The average flow is 207 c.f.s. The maximum flood on record is 21,000 c.f.s. and the minimum flow 1.0 c.f.s. The site is a narrow gorge with massive shist on the right abutment and somewhat less massive shist on the bottom and left abutment. The site would be suitable for a concrete gravity dam but would require extensive excavation to reach a suitable foundation particularly on the left abutment. A supply of earth adequate for an earth dam is not readily available. It is an excellent site for a rock fill dam.

Fig. 22 is a plan of the entire project. The dam is 215 feet high and 775 feet long at the crest. It contains 850,000 cubic yards of rock, 70,000 cubic yards of crushed rock filter and 165,000 cubic yards of compacted earth measured in the dam. The dam creates a reservoir with a surface area of 476 acres and a useful storage of 4,536 acre feet.

Design

The basic design of the Bear Creek project consists of a rock fill dam with an unlined diversion tunnel 1,484 feet long, through the right abutment. The

TABLE 1.—SUMMARY OF SOILS DATA

	Nantahala	Queens Creek	Cedar Cliff	Bear Creek	Wolf Creek	East Fork	Chilowee
(a) Optimum Soil Conditions							
Water content, in %							
Average	21.8	22.1	21.4	22.8	22.6	24.5	25.3
Maximum	28.3	24.0	23.8	30.8	30.5	25.2	27.0
Minimum	16.8	20.0	19.5	17.4	19.0	23.8	24.2
Dry density, in pcf							
Average	98.2	98.4	99.4	97.6	97.9	95.7	95.8
Maximum	105.8	100.2	101.1	108.5	105.0	98.5	98.0
Minimum	88.2	95.0	97.0	85.9	84.0	92.8	94.2
(b) In-Place Soil Conditions							
Water content, in %							
Average	22.6	—	24.7	25.2	23.0	24.9	26.7
Maximum	28.8	—	35.0	39.5	36.1	33.1	33.5
Minimum	17.7	—	15.3	15.2	10.0	16.0	21.2
Dry density, in pcf							
Average	100.0	—	—	96.0	98.0	98.0	94.3
Maximum	109.0	—	—	102.5	108.8	103.0	104.8
Minimum	90.8	—	—	83.1	83.8	90.0	87.1
Permeability, in cm per sec x 10^{-4}							
Average	0.0187	0.0195	0.2513	0.0260	0.1874	0.0100	0.0049
Maximum	0.0762	0.0430	1.8900	0.1880	1.7000	0.0500	0.1760
Minimum	0.0014	0.0027	0.0084	0.0012	0.0014	0.0015	0.0006
Shear friction angle, in degrees							
Average	31.4	31.7	32.7	31.8	30.6	30.9	30.5
Maximum	35.0	34.3	36.0	34.3	33.1	33.4	32.0
Minimum	27.9	28.5	29.5	28.8	27.7	28.0	29.0
Cohesion, in psf							
Average	340	403	710	375	455	420	850
Maximum	800	600	880	735	680	890	1100
Minimum	0	200	540	0	180	230	700
(c) General Soil Characteristics							
Specific gravity	2.77	2.79	2.76	2.75	2.78	2.78	2.82
Mechanical analysis							
15% size, in mm							
Average	0.0040	0.0011	0.4900	0.0016	0.0015	0.0032	0.0020
Maximum	0.0200	0.0016	0.1300	0.0210	0.0200	0.0150	0.0080
Minimum	0.0001	0.0010	0.0010	0.0007	0.0001	0.0009	0.0002
85% size, in mm							
Average	0.343	0.466	1.950	0.412	0.390	0.432	0.200
Maximum	0.950	1.800	4.000	1.520	1.900	2.500	0.600
Minimum	0.170	0.036	0.650	0.059	0.040	0.110	0.150
(d) Performance							
Vertical settlement, in ft	2.85	0.28	1.08	0.95	0.44	0.43	0.16
Vertical settlement, as % of height	1.12	0.36	0.65	0.43	0.27	0.32	0.21
Horizontal movement, in ft	1.53	0.18	0.26	0.54	0.28	0.16	0.12
Years of settlement observation	15.2	8.3	5.5	3.8	2.8	2.8	0.8
Leakage, in cfm	4.74	3.1	4.3	8.0	14.6	2.67	—

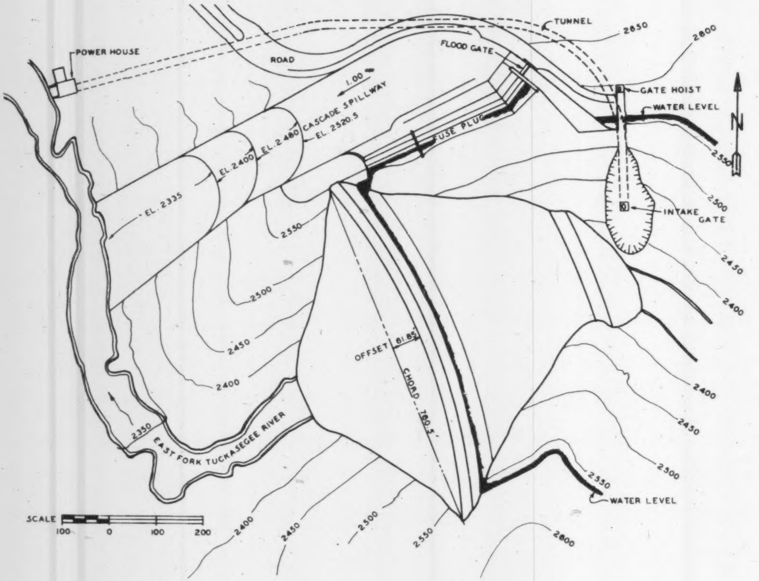
TUNNEL
POWER HOUSE
ROAD
FLOOD GATE
2850
2600
GATE HOIST
WATER LEVEL
1.00 %
CASCADE SPILLWAY
EL. 2520.5
EL. 2480
EL. 2400
EL. 2335
FUSE PLUG
2550
INTAKE GATE
2500
2450
2400
OFFSET 61.85'
CHORD 790.5'
2350
EAST FORK TUCKASEGEE RIVER
2400
2450
2500
2550
WATER LEVEL
SCALE
100
0
100
200
2400
2450
2500
2550
2600

FIG. 22.—PLAN OF BEAR CREEK DAM

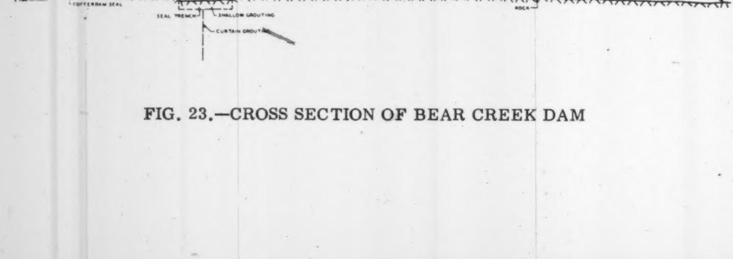
EL. 2580 MAX. CREST
EL. 2570 ABUTMENT CREST
WATER LEVEL EL. 2560
QUARRY RUN ROCK
(+½-3)CRUSHED ROCK
(-½)SAND
IMPERVIOUS EARTH
SAND (-½)
CRUSHED ROCK (+½-3)
CRUSHED ROCK (+3-10)
MAIN ROCK FILL
QUARRY RUN ROCK
SMALL
MEDIUM
LARGE
20'
20'
23'
COFFER DAM
QUARRY RUN ROCK
SMALL
LARGE
UPSTREAM ROCK TOE
QUARRY RUN ROCK
SMALL
MEDIUM
LARGE
COFFERDAM SEAL
SEAL TRENCH
SHALLOW GROUTING
CURTAIN GROUTING
ROCK

FIG. 23.—CROSS SECTION OF BEAR CREEK DAM

lower end of the diversion tunnel to be lined and used as a power conduit. A 10,000 KW hydroelectric unit is installed in a small powerhouse at the exit end of the diversion tunnel. On the right abutment there is a cascade spillway with a capacity of 60,000 c.f.s. controlled by one 25 feet by 25 feet tainter gate and a fuse plug.

The design of a rock fill dam should take into account the conditions of the site. The character and accessibility of the available material and the construction procedures, to achieve the optimum economy.

Measured in the cut, the rock will swell approximately 35% in a dumped rock fill. The spillway was designed to furnish exactly the rock required for the dam. This resulted in a cascade spillway with three cascades. The velocity in the spillway was limited to 20 feet per second. The cascades were designed to provide a definite quantity of rock at a definite elevation to facilitate hauling to the dam.

Measured in the cut the compacted earth core shrank 17%.

Fig. 23 shows a maximum section of the Bear Creek Dam and Fig. 24 shows a detail of the top part of the dam.

In this mountainous country there is very little soil but a diligent search found enough earth for the impervious core within a mile of the dam. The filters were made of crushed and screened rock since there was no sand or gravel available.

The construction of the diversion tunnel was started first. While it was being driven the dam site was cleared and loose material removed. When the diversion tunnel and intake had been completed the upstream cofferdam was built using rock from the lowest cascade. The seal trench was then excavated and grouted and the first section of the main rock fill was built taking rock from the second cascade. The small upstream section of the main rock fill was adopted for 4 reasons.

(1) It permitted the completion of the seal trench without interfering with dumping the main rock fill.
(2) It used rock from a cascade at the proper elevation.
(3) It permitted completing the upstream part of the dam to Elevation 2435 during a dry period.
(4) It provided a much higher cofferdam to protect the work from floods.

During the construction period a flood did occur which overtopped the cofferdam but did not overtop the initial section of the main rock fill and did no damage.

Fig. 25 shows the grain size specified for the rock filter and sand filter and the actual grating of these filters and of the impervious earth. The coarse rock filter (3" to 10") was used to smooth up the irregularities in the dump rock fill. It was dumped from the top of the dam. The fine rock filter, the sand filter and impervious earth was placed in one operation at the same elevation. Approximately one foot thickness of the filter layers migrated into the voids below during the placing operation.

Impervious material was hauled to the site in trucks, spread by bulldozers in 8" layers and compacted with sheep's foot rollers. Care was exercised to be sure that no core was placed at less than the optimum water content. The operation of equipment on the impervious fill set the limit to the maximum amount of water permissible in the core material. The surface of the impervious fill was sloped at all times so that rain water would readily drain off the fill. The upstream filter and the upstream rock blanket are kept to

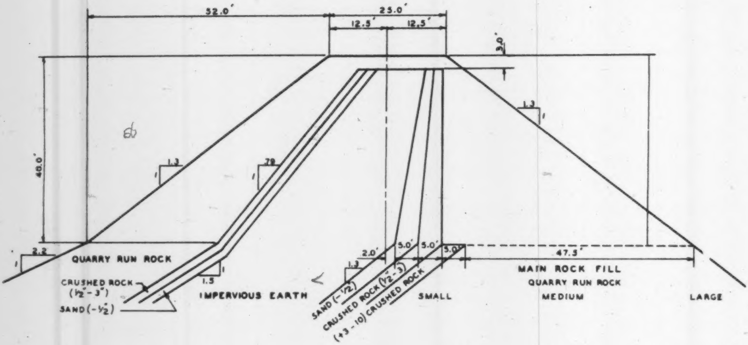
52.0'
25.0'
12.5'
12.5'
3.0'
1.3
1
48.0'
1.3
1
.79
1
2.2
1
QUARRY RUN ROCK
1.5
1
CRUSHED ROCK (½"-3")
SAND (-½")
IMPERVIOUS EARTH
2.0'
5.0'
5.0'
5.0'
1.3
1
SAND (-½)
CRUSHED ROCK (½"-3)
(+3-10) CRUSHED ROCK
47.5'
MAIN ROCK FILL
QUARRY RUN ROCK
SMALL
MEDIUM
LARGE

FIG. 24.—CREST DETAILS OF BEAR CREEK DAM

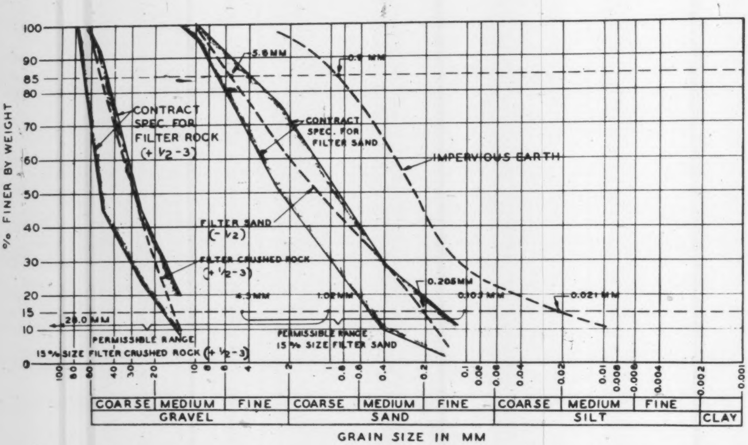

FIG. 25.—TYPICAL GRADING CURVES FOR EARTH AND FILTER MATERIALS FOR BEAR CREEK DAM

within a few feet of the top of the impervious fill as it was placed. This prevented washing the impervious fill during heavy rains.

Performance

Bear Creek Dam was closed October 8, 1953 and the reservoir was substantially filled by January 1954. Delay in installation of the hydroelectric unit made it necessary to use the spillway for several weeks discharging a maximum of 5,000 c.f.s. As noted previously a vertical settlement has been 0.95 feet and the horizontal 0.54 feet. The rate of settlement has decreased so that very little settlement is expected to occur in the future. The small settlement of approximately one-half per cent of the height of the dam is primarily due to placing the basic rock fill in one lift rather than in several lifts, as was done at Nantahala: and to a very thorough job of sluicing the rock fill as it was dumped.

The rock in the seal trench was reasonably impervious and only a moderate amount of grouting was done. No attempt was made to be absolutely sure that no water passed through the foundation. A perfect job of grouting would have cost more than it was worth. The leakage of 8.0 cubic feet per minute is very small for a dam of this size and height. It is believed that all of it occurs through the foundation rather than through the impervious core.

Since the reservoir filled on November of 1953 it has been drawn down approximately 25 feet on several occasions. Generally it has operated within 2 or 3 feet of full. The leakage has remained relatively constant except during rainy weather when the water from a substantial drainage area is included in the measured amount. No maintenance of any kind has been required. There is reason to believe that no maintenance of the dam will be required in the foreseeable future.

Economy

The actual cost of the Bear Creek Dam including the spillway has been compared with the estimated cost of the concrete gravity dam creating a reservoir of the same size. A rock fill dam cost less than one-third of the estimated cost of a concrete gravity dam.

ACKNOWLEDGMENTS

Information with respect to the Kenney Dam was furnished by the International Engineering Company of San Francisco, California with the approval of Mr. McNeely DuBose, Vice President of the Aluminum Company of Canada. The International Engineering Company designed the Kemono Hydroelectric Development including the Kenney Dam. The information with respect to the Nantahala, Queens Creek, Cedar Cliff, Bear Creek, Wolf Creek, and East Fork Dams was obtained from the Hydraulic Engineering Department of the Aluminum Company of America with the assistance of Mr. B. J. Fletcher, Chief Hydraulic Engineer and Chief Assistant Engineer of Alcoa. The filter and soil characteristics were supplied by Mr. H. C. Sperry, Engineer, Hydraulic Department, Alcoa who had charge of the soils mechanics investigations for each of the 6 Alcoa Dams.

DISCUSSION

TORALD MUNDAL,[3] **F. ASCE.**—The author points out that the design of a rockfill dam with sloping earth core is quite simple. The main rock fill supporting the core and the waterload will provide ample factors of safety if it is dumped at the natural angle of repose. This has been verified by laboratory experiments carried out at the University of California for International Engineering Company. It was concluded from these experiments that a triangular rock fill has a large factor of safety against shearing failure when the fluid pressure is applied to the upstream sloping face of the fill.

Model tests were subsequently carried out by the same laboratory to investigate the earthquake resistance of rockfill dams.[4] It was concluded from these experiments that rockfill dams with clay core are inherently very resistant to earthquakes. It was also indicated that the sloping core type is somewhat more earthquake resistant than the central core type because of its greater rigidity.

The general stability of a sloping core rockfill dam is ordinarily not questioned, but there are details of both design and construction which require very careful consideration and treatment. An appreciable settlement can not be avoided in a dumped rockfill. In order to avoid damage to the core, large abrupt differential settlements must be prevented. The materials going into the main rock fill should, therefore, have uniform settlement characteristics and extensive pockets of inferior materials, particularly concentrations of fine fragments, should not be allowed.

The contact between clay core and abutments requires special care. The abutments should be shaped in such a way that the change in the amount of settlement is gradual. Close control of construction operations, by personnel experienced in this type of work, can not be over emphasized.

The author, who originated large sloping clay core rockfill dams, has incorporated in the Bear Creek Dam the improvements gained in his later extensive experience with this type of dams. This dam is an excellent example of obtaining maximum economy by close coordination of design with construction methods. When these principles are adhered to, a sloping core rockfill dam on a suitable site will compare favorably costwise with other types.

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[3] Vice-Pres. and Chf. Engr., International Engineering Co., San Francisco, Calif.

[4] "Earthquake Resistance of Rockfill Dams," by Roy W. Clough and David Pirtz, Transactions, ASCE, Vol. 123, 1958, pp. 792-816.

JOHN LOWE, III,[5] F. ASCE.—The design and construction principles presented by Mr. Growdon, based on his experience with six sloping core rockfill dams, provide valuable information to engineers concerned with this type of dam. The steep slopes used are particularly impressive.

A discussion has been prepared which briefly outlines the design of the Hirfanli Dam in Turkey and the Binga Dam in the Philippines, both of which are of the sloping core rockfill type. In both, rolled rockfill was used immediately downstream of the impervious core instead of the dumped rockfill used in the six dams described by the author. Information on these two dams is given with the idea that it may supplement the valuable data presented on dams with sloping earth cores.

HIRFANLI DAM

The Hirfanli project provides power, flood control, and irrigation benefits. The project consists of a rockfill dam, ungated spillway, vertical tower intake, power tunnels, penstocks, and a powerhouse. The site is located on the Kizil Irmak River, approximately 80 airline kms southeast of Ankara.

The layout of the dam and appurtenant works is shown in Fig. 26. The height of the dam is 80 m and its crest length 364 m. The spillway has a concrete ogee weir with a crest length of 108 m located in a cut in the right abutment. Two short power tunnels pass through the right abutment; portions of these together with a low level extension were used for diversion. Three 46,000 kva generators, each with Francis turbines as prime movers, are housed in the reinforced concrete powerhouse located at the toe of the dam. Provision has been made for future installation of a fourth unit.

The thickness of the alluvial overburden in the riverbed varied from about 2.5 m to 12 m. Generally, sound gabbro bedrock was found below the alluvium, except for one region on the axis of the dam where a 15 m thick zone of weathered material existed below the overburden. A shallow cover of weathered gabbro existed on the abutments and graded into sound gabbro with depth.

Materials available for embankment construction were sound to highly weathered gabbro from spillway excavation, and sandy, clayey silt from borrow in the flood plain upstream of the dam.

The sound gabbro found in the spillway excavation was hard and dense and had a specific gravity exceeding 3.10. This gabbro was used in the downstream

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[5] Assoc. Partner, Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

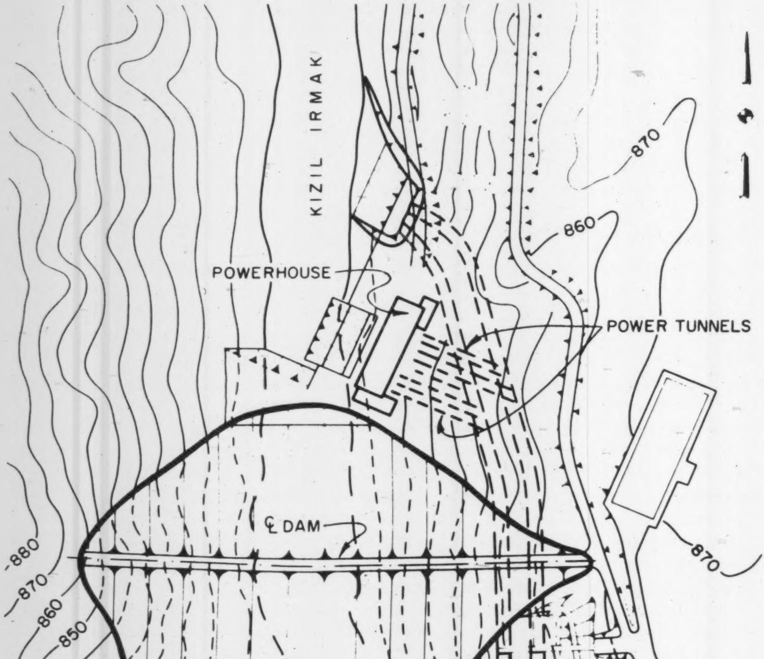
KIZIL IRMAK
POWERHOUSE
POWER TUNNELS
℄ DAM
POWER INTAKE
DIVERSION TUNNELS
DIVERSION TUNNEL INTAKES
FLOW
880
870
860
850
840
830
820
810
800
790
870
860
870
850
840
830
820
810
790
800
0
100
SCALE IN m.

FIG. 26.—PLAN OF HIRFANLI DAM

shell of the dam as sluiced dumped sound rockfill. Much of the excavated gabbro was disintegrated or weathered. The weathered gabbro was used as rolled fill in the center section of the dam immediately downstream of the core. Free draining, coarser, disintegrated gabbro, with little or no fines, was used in the rolled upstream shell of the dam.

Insufficient quantities of natural sand and gravel were available for filters. For this reason sound unweathered gabbro, excavated by selective digging in the spillway cut, was used.

The design of the embankment was influenced by the available quantities of each type of rock and by the order in which these quantities became available during excavation. From preliminary excavation it was estimated that approximately one-third of the total quantity of the spillway excavation would consist of sound rock. In the design of the embankment, therefore, the volume of sound rock was kept to a minimum. As the excavation proceeded, the design was adjusted to the quantities of material of each type actually available, by varying the line of demarcation between the central weathered rock zone and the downstream sound rock zone. The quantities of the structural excavation closely balanced the requirements of the embankment.

During the design stage both a rockfill dam with a vertical impervious core and one with a sloping impervious core were considered. However, the sloping core dam better fitted the topography and foundation conditions. If a central core dam were placed so its core were located at the same place as the sloping core where the depth of alluvium in the river were least, a much larger volume of embankment would be required due to widening of the valley upstream. A cross section of the dam is shown in Fig. 27. The availability of weathered gabbro permitted construction of a large rolled rockfill section immediately downstream of a sloping core. The use of such a section of rolled fill minimizes settlement of the core. It is, therefore, considered by several members of the Board of Consultants and the writer, as a requirement in the design of a sloping core dam in spite of the fact that the author has had good success in constructing sloping core dams without such a feature.

The volume of the materials in the embankment is as follows:

Sound rockfill	866,000 cu m
Weathered gabbro	942,000 cu m
Impervious core	122,000 cu m
Filter material	99,000 cu m
Impervious blanket (cofferdam)	5,000 cu m

The stability of the dam was analyzed for four possible conditions of operation, namely: immediately after construction; static pool at various elevations; effect of earthquake with the reservoir at critical pool elevation; and rapid drawdown from spillway design flood elevation (856.5 m) to the invert elevation of the power intake (818.0 m).

The condition of rapid drawdown was analyzed using a new modification of the Modified Method of Slices.[6] The new method considers the effect of anisotropic consolidation on shear strength. Application of the new method to Hirfanli Dam is described elsewhere.[7] The determination of the shear strength

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[6] "Earth Dams," by T. T. Knappen and John Lowe, III, Handbook of Applied Hydraulics, Calvin Davis, ed. (second edition), John Wiley & Sons, 1952.

[7] "Stability of Earth Dams Upon Drawdown," by John Lowe, III and Leslie Karafiath, First Panamerican Conf. on Soil Mechanics and Foundation Engr., Mexico City, Mexico, September, 1959.

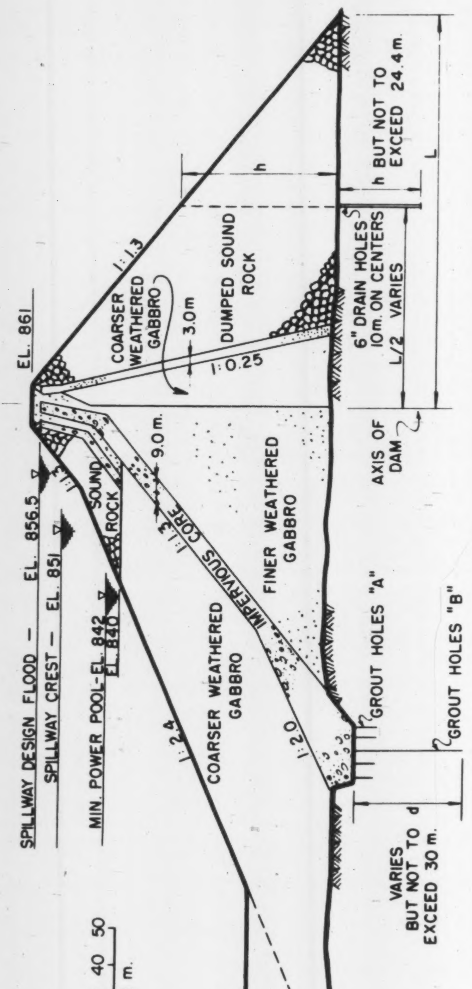
SPILLWAY DESIGN FLOOD — EL. 856.5
SPILLWAY CREST — EL. 851
MIN. POWER POOL-EL. 842
EL. 840
EL. 861
℄
SCALE IN m.
0 10 20 30 40 50
EL. 800
WEATHERED GABBRO
1:2.4
COARSER WEATHERED GABBRO
1:2.0
IMPERVIOUS CORE
1:1.3
SOUND ROCK
9.0 m.
FINER WEATHERED GABBRO
COARSER WEATHERED GABBRO
3.0 m
1:0.25
DUMPED SOUND ROCK
1:1.3
h
GROUT HOLES "A"
GROUT HOLES "B"
VARIES BUT NOT TO EXCEED 30 m.
d
AXIS OF DAM
6" DRAIN HOLES 10 m. ON CENTERS
L/2 VARIES
h BUT NOT TO EXCEED 24.4 m.
L

FIG. 27.—CROSS SECTION OF HIRFANLI DAM

for anisotropic consolidation condition for Hirfanli Dam has also been described elsewhere.[8]

Foundation treatment consisted of a grout curtain and drain holes. The depth of grout curtain is equal to approximately one-third the height of the dam. A shallow grout platform was provided under the full width of the core to assure good contact between the core and the curtain. Vertical drain holes were placed at the downstream one-quarter point of the dam to relieve possible water pressures in the downstream section of the foundation.

After construction was completed, a crack developed along the crest between downstream dumped sound rockfill and the filter. The crack is attributed to the dumped rockfill settling more than the rolled rockfill and is not considered serious.

The crack is similar to those mentioned by Mr. Growdon as occurring at Cedar Cliff and Bear Creek dams and attributed to differential settlement. A second possible cause for the cracks mentioned by the author, however, is that the upstream rockfill does not receive hydraulic pressure whereas the earth core behind it does. Thus, the portion of the embankment downstream of the core, shifted downstream due to hydraulic pressure on the core, but the upstream rockfill did not. The curvature of the dams also contributed to the upstream shell not following the horizontal movement of the remainder of the dam.

Acknowledgments.—Hirfanli Dam was designed by Tippetts-Abbett-McCarthy-Stratton for the General Directorate of State Hydraulic Works of the Turkish Government. It was constructed by George Wimpey & Co., Ltd., and the English Electric Co., Ltd., under supervision of the designer and the Turkish Government. Mr. Growdon, Arthur Casagrande, Hibbert Hill, and Friedrich Tolke composed the Board of Consultants.

BINGA DAM

The Binga Project is an integral part of the planned development of the Agno River in Luzon, Republic of the Philippines. It is operated in conjunction with the Ambuklao Reservoir upstream to obtain the optimum output for both power plants. Whereas the Ambuklao Dam has a relatively large storage reservoir, the Binga Dam has a small reservoir limited primarily to regulation of the releases from Ambuklao. The Binga Dam project is primarily a power project and develops a gross head of 154 m with an installed capacity of 100,000 kw. The project consists of a rockfill dam, two diversion tunnels, a gated spillway, a power intake and pressure tunnel, an underground powerhouse, and a tailrace tunnel.

The height of the dam is 103 m with a crest length of 210 m. Two horseshoe shaped diversion tunnels, 7.6 m in diameter, are driven through the left abutment. The spillway is located in a deep side hill cut on the left abutment. It has six tainter gates each 12.5 m wide by 12.0 m high and can pass a maximum flood of 10,000 cms. The power intake is in the right abutment; a pressure tunnel 800 m long and 6 m in diameter leads to the surge tank and steel lined penstock shafts. A 6 m diameter tailrace tunnel, 2,000 m long, returns the water from the underground powerhouse to the river. The general layout of dam and vicinity is shown in Fig. 28.

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[8] "Effect of Anisotropic Consolidation of Undrained Shear Strength of Compacted Clays," by John Lowe, III and Leslie Karafiath, ASCE Research Conf. on Shear Strength of Cohesive Soils, Boulder, Colorado, June, 1960.

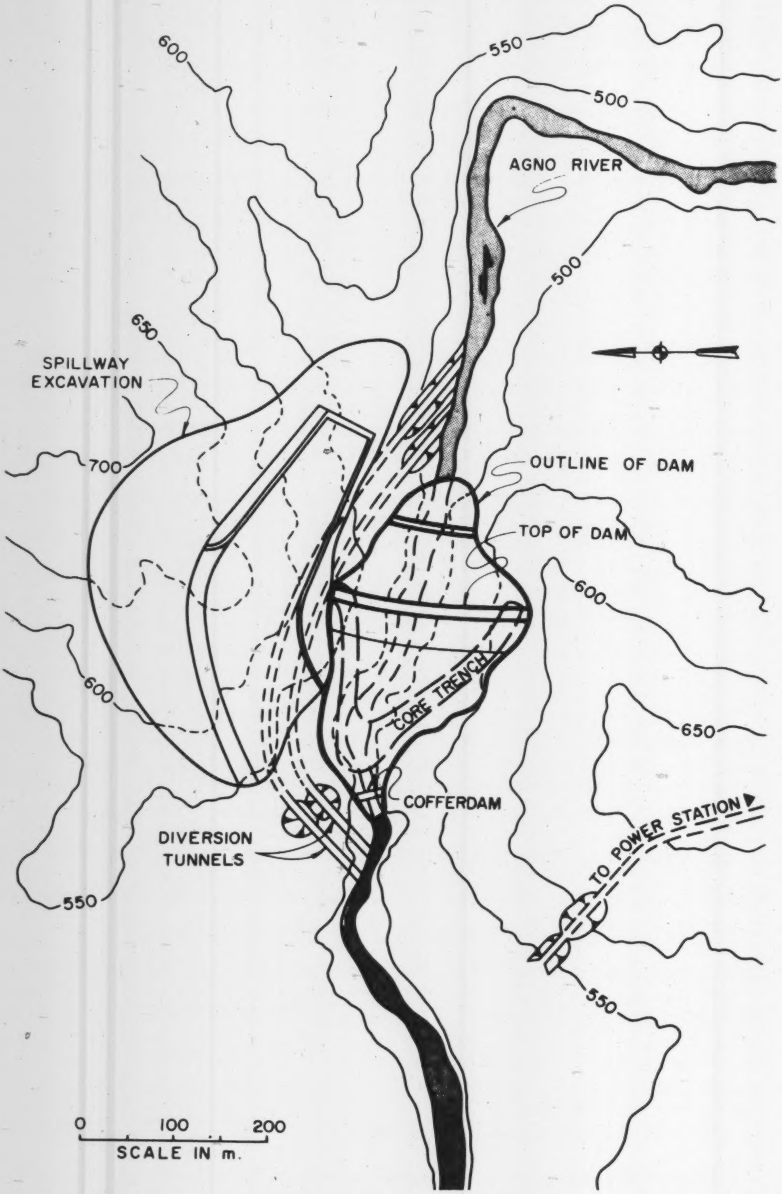
600
550
500
AGNO RIVER
500
650
SPILLWAY
EXCAVATION
700
OUTLINE OF DAM
TOP OF DAM
600
CORE TRENCH
600
650
COFFERDAM
TO POWER STATION
DIVERSION
TUNNELS
550
550
0
100
200
SCALE IN m.

FIG. 28.—PLAN OF BINGA DAM

The dam was built during two construction seasons which run from January through June. During the first construction season it was proposed that a cofferdam be placed, the river diverted through the tunnels, and the dam constructed to El. 569. Four methods of controlling floods between construction seasons were available in this plan: the two diversion tunnels; storage in Binga Reservoir to El. 569; storage above Ambuklao spillway crest; and use of the unfinished Binga Spillway excavated to El. 557.

Originally the formations at the site were lava flows, volcanic ash, and water-deposited sediments laid down in Tertiary time. Subsequently these rocks were metamorphosed with heat, pressure, and folding so that the original structure of the rocks was generally not discernable except on the left abutment where the stratifications of the rocks could be seen dipping upstream. Many fractures and faults resulted from the folding. The metamorphic series was then subjected to the injection of molten diorite and other igneous rocks. These injections appeared as dikes and widely distributed veinlets in the fractures. The faults at the proposed dam site were minor and generally occurred before the injection of the igneous rocks. There was no indication of movement on them.

Weathering of the rocks at the site was generally deep but did not extend below ground water table. Weathering was greater on the right abutment than on the left because a set of minor faults running SE intersected a set running SW in that area. At higher elevations there was present on both abutments residual soil resulting from disintegration and decomposition of the parent rocks. Because of the tropical climate these soils contained a preponderance of iron oxides due to leaching out of silica. Between the residual soil cover and unweathered rock a transition zone of broken soft rock existed. The unweathered rock was hard but brittle and generally broken. It varied from being highly broken rock to occasional relatively sound massive rock at depth.

Only a small amount of sound and massive rock was present. The rock was exposed in the river valley and in much of the left abutment. The upper part of the left abutment and most of the right abutment had only a thin mantle of overburden soil and transitional soft rock overlying the hard, but brittle and broken, bed rock. In the fault areas the rock was much more broken than in between the faults. To minimize grouting problems the impervious core was designed to contact the foundation in between faults.

In the spillway excavation the major material produced was hard, but broken, rock suitable for rockfill. About 80% of the material was less than the 30 cm size and was fairly well graded. It was suitable for placement as rolled rockfill in 45 cm layers provided the plus 30 cm material was raked out. The raked out material was suitable for dumped rockfill. A small percentage of the excavation was in sound relatively massive rock and this rock produced material directly suitable for sluiced dumped rockfill.

A deposit of sand and gravel was found as a terrace near the dam site and was suitable for filters on either side of the impervious core of the dam. The material was well graded and compacted well.

The spillway excavation produced two materials which were suitable for the impervious core, namely the soil cover from the upper elevations and this same soil cover mixed with some of the disintegrated rock that occurred below it. The mixture was the better material since it had a somewhat higher shear strength. Stockpiling of this material would have been necessary, however, and in order to avoid wetting of the stockpiled material during the rainy season, it was decided to borrow the core material directly from an area in the vicinity of the power intake. The core material borrowed from this area consisted of disintegrated metamorphic rocks.

The scheme of diversion proposed consisted of diverting the river through two horseshoe shaped tunnels and constructing the dam to El. 569 during the first dry season. This height of pool developed sufficient capacity in the two diversion tunnels and in the incompleted spillway to handle a one in thirty year flood. The type of first stage construction which involved minimum volume was a rockfill section with angle of repose slopes upstream and downstream of 1 on 1.4. In order to be stable with pool at El. 569 and upon drawdown after a flood, the rockfill cross-section was constructed in a particular manner. For stability under full pool, the permeability of the upstream portion of the rock had to be less than that of the downstream portion. If this were not so, the force of water seeping through the dam would have caused stability failure downstream. The majority of the rock from the spillway excavation was placed as rolled rockfill and on this basis the upstream face of the first stage construction was made of a 15 m wide zone finer size rolled rockfill, the middle part of coarser sized rolled rockfill, and the downstream third of large sound rock sizes suitable for dumped rockfill.

During the second construction season the sloping core was continued on top of the upstream face of the rockfill resulting from the first stage construction and an upstream rock shell was placed to cover the sloping core. Since the rock dips in an upstream direction, according to exposures on the left abutment, the sloping core had the additional advantage of tying in better to the foundation than a vertical core. The horizontal thickness of the impervious core at spillway crest level was 10 m and at valley bottom 30 m. This thickness of core resulted in a maximum hydraulic gradient of 5 for reservoir at normal pool level. A cross section of the dam is shown in Fig. 4.

Filters were placed between the downstream rockfill and the core and between the core and upstream rock shell. This material consisted of sand and gravel from the terrace near the dam site. Construction of the upstream rock shell was accomplished in a manner similar to that used for the downstream rock shell with the finer rolled rock placed adjacent to the filter layer on the upstream side of the impervious core. The filter layer downstream of the core was 3 lanes (7.5 m) wide and the upstream 2 lanes (5.0 m) wide, both up to the crest of the first stage construction.

The slope of the downstream face of the dam was the angle of repose of dumped rock which was 1 on 1.4. The slope of the upstream face below El. 563 was 1 on 2.5. Above El. 563 a center core rockfill dam section with an upstream slope of 1 on 1.4 was used.

The dam was cambered from El. 584 at the abutments to El. 586 at the midpoint of its length. The axis of the dam was curved upstream so that under water load the rockfill downstream of the core tended to wedge between the valley walls due to slight arch action. The curvature was such that the midpoint of the curved axis was 10 m upstream of the chord line drawn between the ends of the dam at crest elevation.

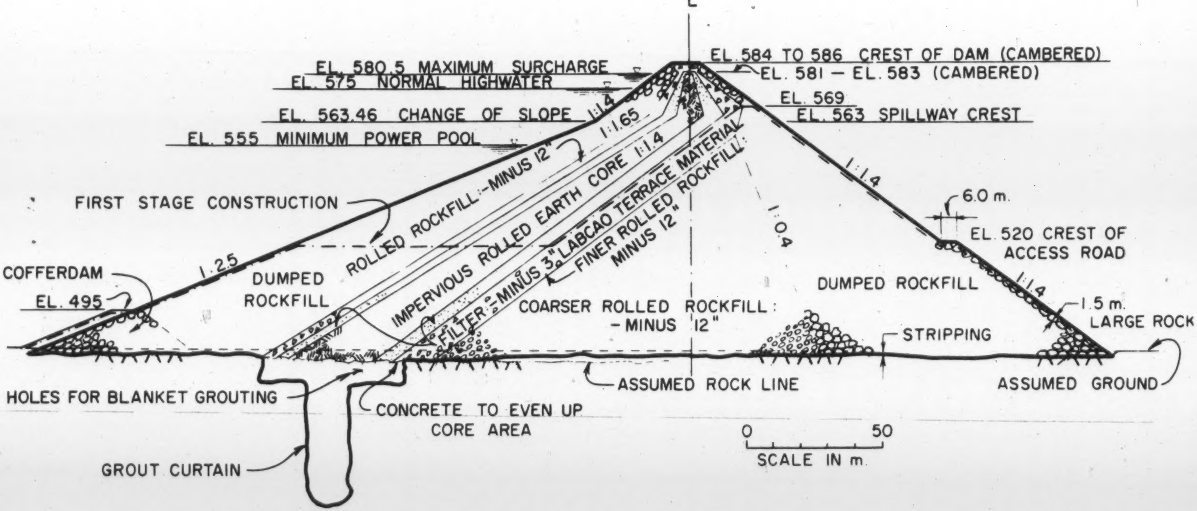
℄
EL. 580.5 MAXIMUM SURCHARGE
EL. 575 NORMAL HIGHWATER
EL. 563.46 CHANGE OF SLOPE
EL. 555 MINIMUM POWER POOL
EL. 584 TO 586 CREST OF DAM (CAMBERED)
EL. 581 – EL. 583 (CAMBERED)
EL. 569
EL. 563 SPILLWAY CREST
1:1.4
1:1.65
1:1.4
1:1.4
1:0.4
1:1.4
1:1.4
1:2.5
FIRST STAGE CONSTRUCTION
ROLLED ROCKFILL:-MINUS 12"
IMPERVIOUS ROLLED EARTH CORE 1:1.4
FILTER:-MINUS 3" LABCAO TERRACE MATERIAL
FINER ROLLED ROCKFILL: MINUS 12"
COARSER ROLLED ROCKFILL: -MINUS 12"
6.0 m.
EL. 520 CREST OF ACCESS ROAD
COFFERDAM
EL. 495
DUMPED ROCKFILL
DUMPED ROCKFILL
1.5 m. LARGE ROCK
STRIPPING
HOLES FOR BLANKET GROUTING
ASSUMED ROCK LINE
ASSUMED GROUND
CONCRETE TO EVEN UP CORE AREA
GROUT CURTAIN
0
50
SCALE IN m.

FIG. 29.—CROSS SECTION OF BINGA DAM

The method of stability analysis for the Binga Dam was essentially the same as that performed for the Hirfanli Dam.

Grouting of the foundation rock was required for tying the impervious core into the foundation. There was no zone of relatively unbroken impervious rock to which the grout curtain could be tied, so the minimum depth of grouting in the valley was to a penetration of 1/2 the head developed under the maximum normal reservoir level but not less than 25 m. In order to tie the grout curtain to the impervious core, the foundation of the core was either excavated to groutable rock and an impervious platform formed by grouting this area using 5 m deep grout holes, or a 0.5 m thick concrete mat was provided. Where open seams appeared at the surface they were filled with concrete from the surface. The row of deep grout holes for the grout curtain were grouted to the platform and the core placed tight against the platform.

Acknowledgments.—The owner of the project is the National Power Corporation of the Philippines. Tippetts-Abbett-McCarthy-Stratton together with the Engineering and Development Co. of the Philippines were consultants to the Corporation during design and construction. Construction was by the Philippine Engineering Syndicate Inc.

JAMES P. GROWDON,[9] **F. ASCE.**—It has been properly pointed out by Mr. Mundal that certain features of a rockfill dam require considerable care during construction. His comments are worth considerable emphasis.

Mr. Lowe's discussion is a brief and adequate description of the design and construction of two sloping earth core rockfill dams.

The optimum in dam design and construction is to build a structure which is safe against all the forces of nature which tend to destroy it, and at the least cost. To accomplish this it is necessary to use correctly materials most readily available.

A dumped rockfill requires sound large rock so that the voids in the fill can absorb the fines which always accompany quarry operations.

The required excavation for the projects described by Mr. Lowe produced some rock suitable for a dumped rockfill and some rock which was unsuitable because of the small sizes and excess fines. The rock unsuitable for dumped rockfill was entirely suitable for a compacted rockfill. The dumped rockfill and the compacted rockfill can be interleaved without the necessity of determining in advance the exact quantities of each class of material which will be available.

Mr. Lowe's discussion illustrates the flexibility of the sloping earth core rockfill type in the hands of a skillful designer. Mr. Lowe's discussion has added materially to the usefulness of the writer's paper.

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[9] Consultant, Aluminum Co. of America, Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3070

ROCKFILL DAMS: PERFORMANCE OF SLOPING CORE DAMS

By James P. Growdon,[1] F. ASCE

With Discussion by Messrs. F. L. Lawton; and James P. Growdon

SYNOPSIS

The design and construction of seven dams is briefly reviewed and performance is discussed measured in terms of leakage, settlement, control and passage of floods, operation, repairs and maintenance. Conclusion reached that sloping core rockfill dams so far designed and constructed have performed to entire satisfaction of owners and operators.

INTRODUCTION

A very brief description of the design and construction of these dams is a necessary preliminary to a report of their performance.

The basic structural element of each of these dams is a dumped rockfill with slopes the natural angle of repose. The rock for two of the dams is sound, hard arkose. The rock used in five of the dams is schist, somewhat softer than the arkose. In all but one case the excavation of a spillway channel through an abutment provided the material with which to build the dam.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1744. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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An impervious earth core on the upstream slope of the basic rockfill provides a watertight membrane. This core is protected on both sides by a graded filter which prevent the migration of the finer particles into the larger voids. The core is protected from wave action and from sloughing into the reservoir by a rock blanket which is free draining.

The earth core is in direct contact with the rock in the bottom and the abutments in a seal trench excavated to sound rock and grouted. The thickness of the earth core is determined by three factors:

1. To eliminate the leakage through the core to an acceptable amount (very small).
2. To permit placing with mechanical equipment.
3. To prevent rupture due to possible differential settlement.

The dams are curved upstream and are cambered so that the center section of the dam is built to a higher elevation than the abutments.

The spillway channel through the rock of the abutment is unlined. In the Nantahala Dam the flood water flows down a steep slope into the river bed well below the toe of the dam. In the other dams the unlined rock spillway channel ends in one or more cascades which return the flood water to the original river channel quite close to the toe of the dam. The spillways are designed in accordance with the TVA flood formula and will pass a flood much larger than any flood which can reasonably be expected ever to occur.

With two exceptions, the spillways are controlled by Tainter gates with a capacity equal to the "Hundred Year" flood and by fuse plugs for the remainder of the spillway designed flood. These fuse plugs are small sloping core rockfill dams which will wash out in sections when overtopped.

GENERAL DATA

Nantahala Dam

The Nantahala Dam is the first dam of this type to be constructed. (Figs. 1 and 2) It is located on the Nantahala River in western North Carolina.

Drainage Area	-	91 sq. miles
Height	-	250 ft.
Length	-	1042 ft.
Total Volume	-	2,265,000 cu. yds.
Useful Storage	-	126,000 ac. ft.

Queens Creek Dam

The Queens Creek is the second of the sloping core rockfill dams to be constructed by Alcoa. (Figs. 3 and 4) It is located on Queens Creek in western North Carolina, close to the Nantahala Powerhouse.

Drainage Area	-	3.6 sq. miles
Height	-	78 ft.
Length	-	382 ft.
Total Volume	-	89,000 cu. yds.
Useful Storage	-	490 ac. ft.

This dam was built on the alluvial material filling the creek valley.

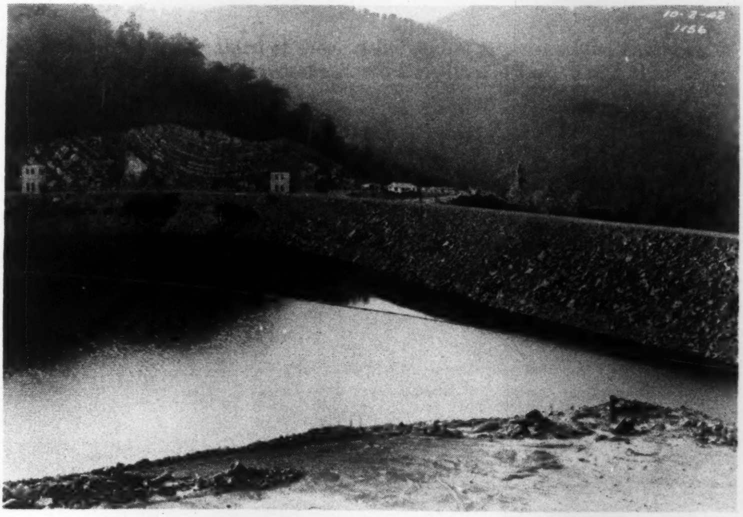
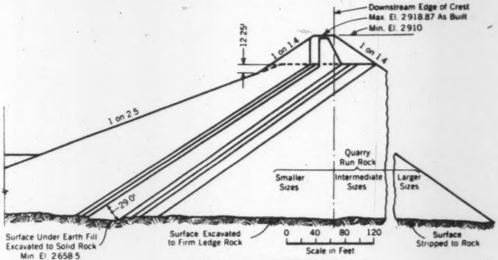
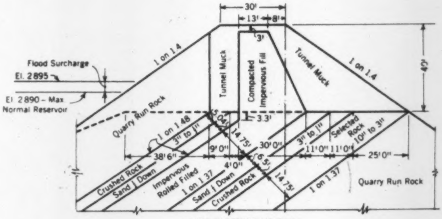


FIG. 1.—NANTAHALA DAM, RESERVOIR FILLED


Downstream Edge of Crest
Max. El. 2918.87 As Built
Min. El. 2910
12.25'
1 on 1.4
1 on 1.4
1 on 2.5
Quarry Run Rock
Smaller Sizes
Intermediate Sizes
Larger Sizes
29.0'
Surface Under Earth Fill Excavated to Solid Rock Min. El. 2658.5
Surface Excavated to Firm Ledge Rock
0
40
80
120
Scale in Feet
Surface Stripped to Rock

(a) TYPICAL CROSS SECTION OF DAM


30'
13'
8'
3'
1 on 1.4
Tunnel Muck
Compacted Impervious Fill
Tunnel Muck
1 on 1.4
40'
Flood Surcharge
El. 2895
El. 2890 - Max Normal Reservoir
Quarry Run Rock
5.04'
3.3'
1 on 1.48
3" to 1"
14.75'
30'0"
3" to 1"
Selected Rock
10" to 3"
38'6"
9'0"
11'0"
11'0"
25'0"
4'0"
6.5'
Crushed Rock
Sand | Down
Impervious Rolled Filled
1 on 1.37
Sand | Down
Crushed Rock
14.75'
1 on 1.37
Quarry Run Rock

(b) CREST DETAILS OF DAM

FIG. 2.—NANTAHALA DAM, CROSS SECTION

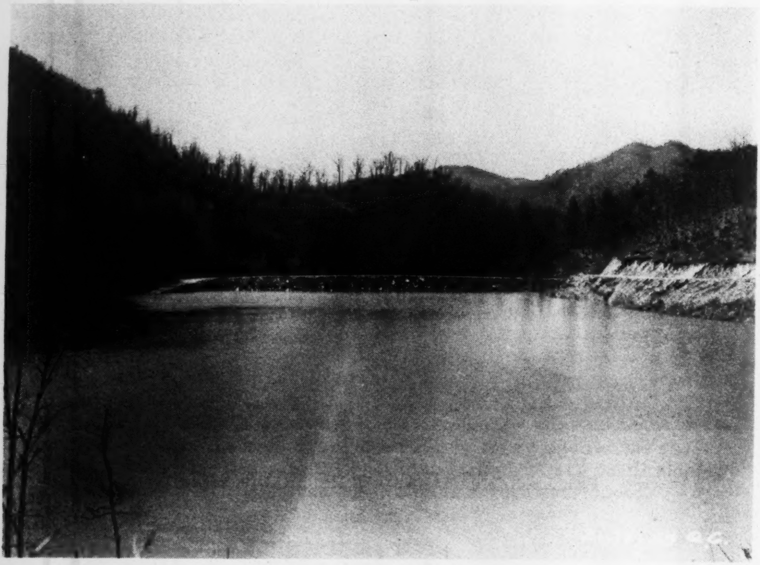


FIG. 3.—QUEENS CREEK DAM, RESERVOIR FILLED

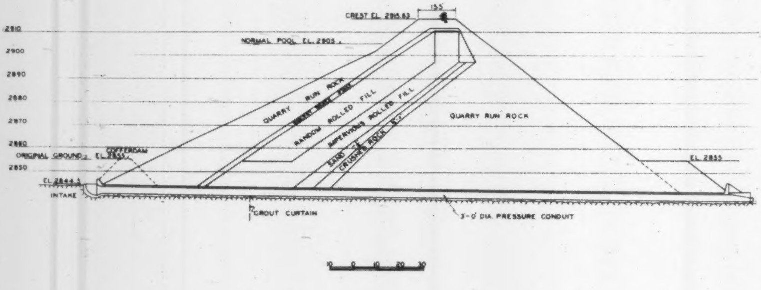

CREST EL. 2915.63
15.5'
NORMAL POOL EL. 2905
2910
2900
2890
2880
2870
2860
2850
QUARRY RUN ROCK
RANDOM ROLLED FILL
IMPERVIOUS ROLLED FILL
SAND
CRUSHED ROCK
QUARRY RUN ROCK
ORIGINAL GROUND EL. 2855
COFFERDAM
EL. 2844.5
INTAKE
GROUT CURTAIN
3'-0" DIA. PRESSURE CONDUIT
EL. 2855
10 0 10 20 30

FIG. 4.—QUEENS CREEK DAM, CROSS SECTION

Cedar Cliff

Cedar Cliff is the third of these dams and is located on the east fork of the Tuckasegee River in western North Carolina. (Figs. 5 and 6)

Drainage Area	-	80.7	sq. miles
Height	-	165	ft.
Length	-	600	ft.
Total Volume	-	686,000	cu. yds.
Useful Storage	-	585	ac. ft.

Bear Creek

Bear Creek is the fourth sloping core rockfill dam built by Alcoa. (Figs. 7 and 8) It is located on the east fork of the Tuckasegee River in western North Carolina, at the upper end of the Cedar Cliff reservoir.

Drainage Area	-	75.3	sq. miles
Height	-	215	ft.
Length	-	775	ft.
Total Volume	-	1,085,000	cu. yds.
Useful Storage	-	4,536	ac. ft.

Wolf Creek

Wolf Creek Dam is located on Wolf Creek a tributary of the east fork of the Tuckasegee River in western North Carolina. (Figs. 9 and 10)

Drainage Area	-	15.2	sq. miles
Height	-	185	ft.
Length	-	810	ft.
Total Volume	-	731,000	cu. yds.
Useful Storage	-	7,640	ac. ft.

East Fork Dam

This dam is on a tributary of the east fork of the Tuckasegee River in western North Carolina. (Figs. 11 and 12)

Drainage Area	-	24.9	sq. miles
Height	-	135	ft.
Length	-	385	ft.
Total Volume	-	195,000	cu. yds.
Useful Storage	-	1,250	ac. ft.

N.B. The Wolf Creek and East Fork Reservoirs are hydraulically connected by a tunnel.

Chilhowee Dam

This is the latest dam constructed by Alcoa. It consists of a concrete gravity spillway and intake with sloping core rockfill abutments. (Figs. 13,

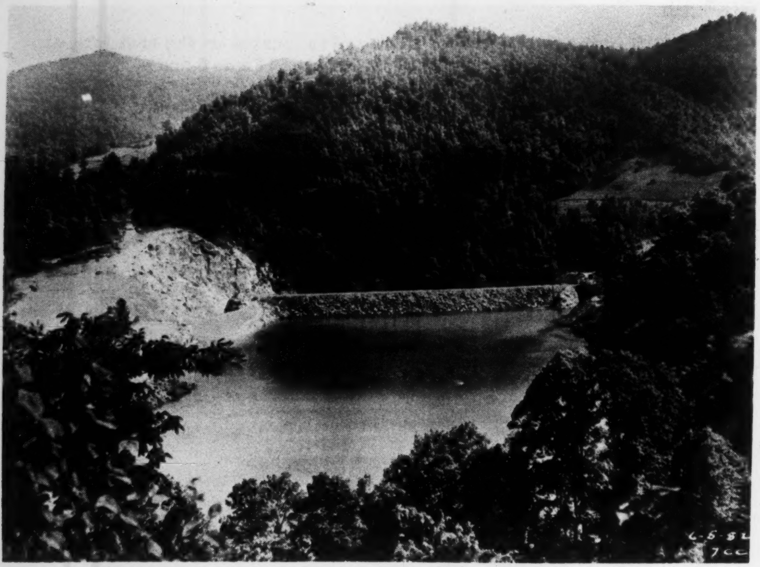


FIG. 5.—CEDAR CLIFF DAM, GENERAL VIEW

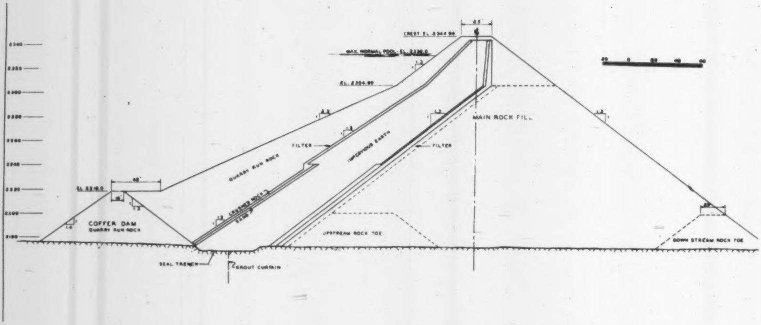

2340
2320
2300
2280
2260
2240
2220
2200
2180
CREST EL. 2344.99
25'
MAX. NORMAL POOL-EL. 2330.0
EL. 2304.99
EL. 2218.0
40'
10'
COFFER DAM
QUARRY RUN ROCK
QUARRY RUN ROCK
CRUSHED ROCK
SAND
FILTER
IMPERVIOUS EARTH
FILTER
MAIN ROCK FILL
UPSTREAM ROCK TOE
DOWN STREAM ROCK TOE
SEAL TRENCH
GROUT CURTAIN
20 0 20 40 60

FIG. 6.—CEDAR CLIFF DAM, CROSS SECTION

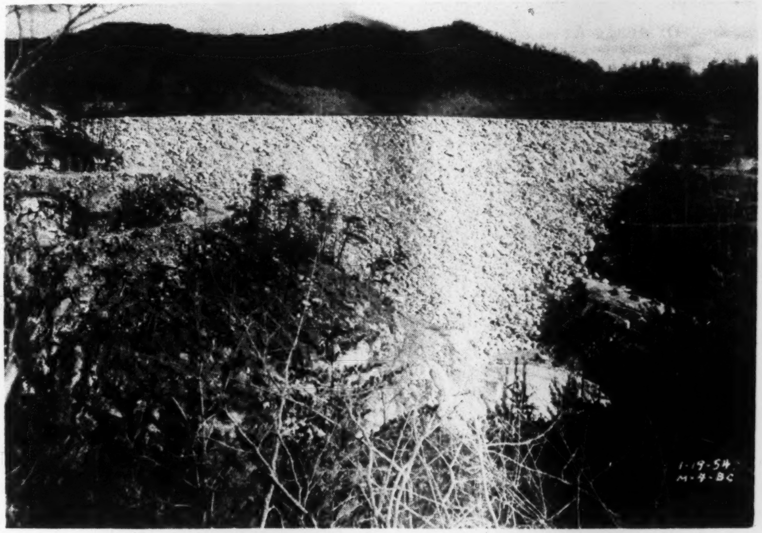


FIG. 7.—BEAR CREEK DAM, DOWNSTREAM FACE

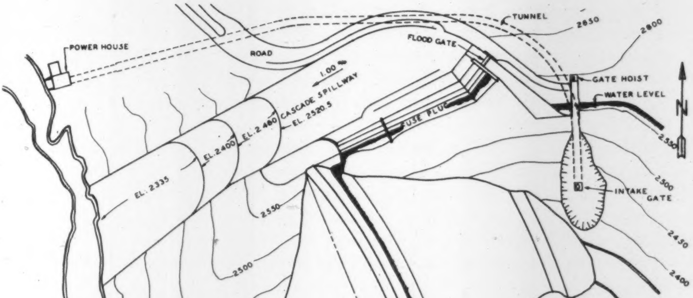

TUNNEL
2650
2600
POWER HOUSE
ROAD
FLOOD GATE
1.00
CASCADE SPILLWAY
EL. 2520.5
EL. 2460
EL. 2400
EL. 2335
FUSE PLUG
GATE HOIST
WATER LEVEL
2550
2500
INTAKE GATE
2450
2400
2550
2500
2450
2400
OFFSET 81.85
CHORD - 780.5'
2400
2450
2500
2550
2350
EAST FORK TUCKASEGEE RIVER
WATER LEVEL
SCALE
100
0
100
200
2400
2450
2500
2550
2600

FIG. 8.—BEAR CREEK DAM, CROSS SECTION

14 and 16) It is located on the Little Tennessee River in eastern Tennessee.

Drainage Area	-	1,977 sq. miles
Height	-	91 ft.
Length of rockfill sections	-	655 ft.
Total Volume	-	353,000 cu. yds.
Useful Storage	-	6,805 ac. ft.

Six Tainter Gates have a discharge capacity of 230,000 c.f.s. with a 5 ft. surcharge.

PERFORMANCE

The performance of a dam may be measured in terms of leakage, settlement, control and passage of floods, operation, repairs and maintenance.

Nantahala

Diversion closed February, 1942—Reservoir filled July, 1942. The reservoir has been substantially emptied and filled each year.

A weir was constructed downstream from the dam to measure leakage. The water measured included the runoff from a 12 acres drainage area and two small springs downstream from the grout curtain in addition to the leakage through the foundation and through the earth core. The water passing the weir was measured frequently until January 1950 at which time the weir was rendered inoperative by flood water discharge through the spillway. With a full reservoir in dry weather the water measured at the weir amounted to 36 gallons per minute. During rains this increased to as much as 135 gallons per minute. There is no evidence that the leakage past the dam has increased since the reservoir was first filled. It is believed that all of the water which passes through the earth core is evaporated before it can be measured and that all of the measured water is surface drainage, the flow of the two small springs and the leakage through the foundation grout curtain.

Since the dam was first closed the maximum settlement has been 2.85 ft. vertical and 1.53 ft. horizontal. The settlement has been a smooth curve from the abutments to the highest section of the dam. There has been no measurable differential settlement. A graph of the settlement vs. time shows that the rate of settlement has decreased so that it will soon be too small to measure. (Fig. 15)

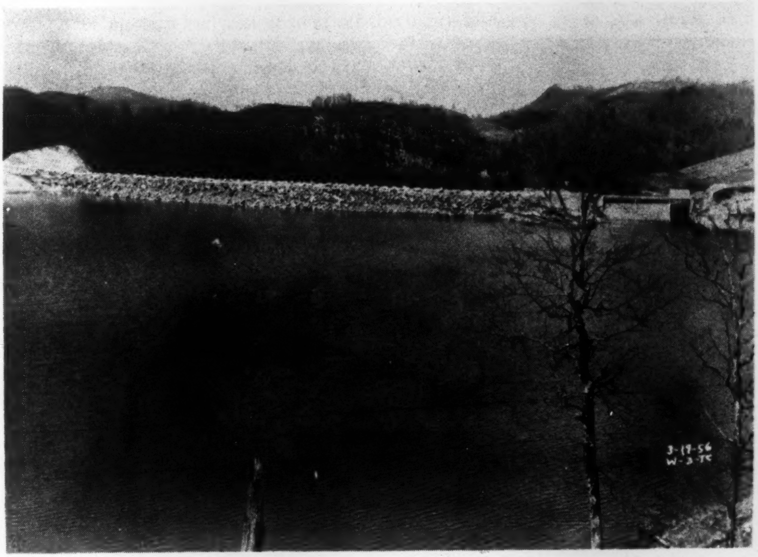


FIG. 9.—WOLF CREEK DAM, RESERVOIR PARTIALLY FILLED

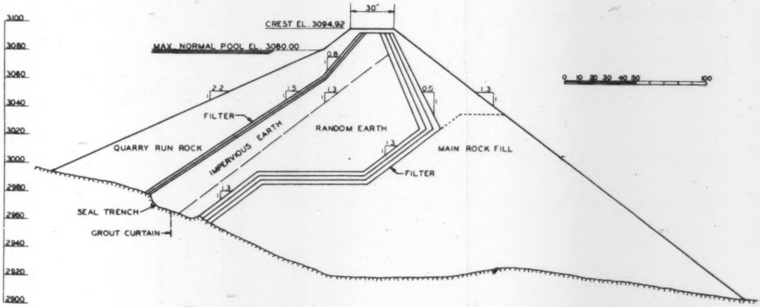

30'
CREST EL. 3094.92
MAX. NORMAL POOL EL. 3080.00
0.8
2.2
1.5
1.3
0.5
1.3
FILTER
IMPERVIOUS EARTH
RANDOM EARTH
QUARRY RUN ROCK
MAIN ROCK FILL
1.3
FILTER
1.3
SEAL TRENCH
GROUT CURTAIN
3100
3080
3060
3040
3020
3000
2980
2960
2940
2920
2900
0 10 20 30 40 50 100

FIG. 10.—WOLF CREEK DAM, CROSS SECTION

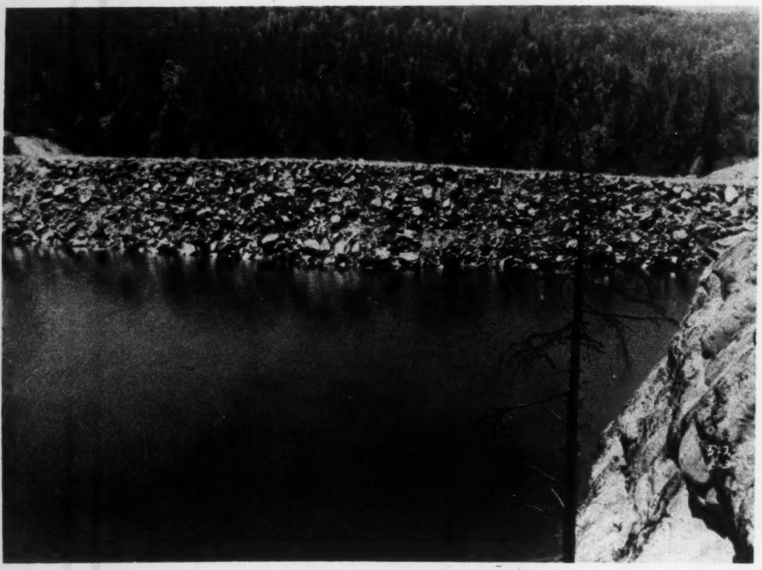


FIG. 11.—EAST FORK DAM, RESERVOIR PARTIALLY FILLED

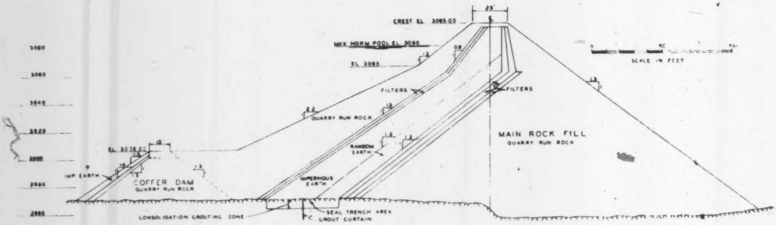

25'
CREST EL
MAX NORM POOL EL
SCALE IN FEET
FILTERS
FILTERS
QUARRY RUN ROCK
MAIN ROCK FILL
QUARRY RUN ROCK
RANDOM EARTH
IMPERVIOUS EARTH
IMP EARTH
COFFER DAM
QUARRY RUN ROCK
SEAL TRENCH AREA
GROUT CURTAIN
CONSOLIDATION GROUTING ZONE

FIG. 12.—EAST FORK DAM, CROSS SECTION

On three separate occasions flood water has passed through the spillway for periods of several days each. Maximum discharge was 3,200 c.f.s. At the lower end of the spillway channel the water flowed down the steep slope to the original stream bed and eroded all the loose material from this slope, as it was expected to do. No erosion occurred in the unlined spillway channel.

Spillway flow is controlled by four Tainter gates and a short section of fuse plug. One of these gates is automatically controlled by a float mechanism.

Opening and closing the spillway gates and operation of the gates controlling the flow of water into the conduit to the turbine are the only operating procedures which have been required at the Nantahala Dam.

In the sixteen years of operation no repairs whatever have been made to the dam. No maintenance, whatever has been required and it is believed that no maintenance except painting the spillway gates will be required in the foreseeable future.

Queen's Creek

Completed in 1948. Reservoir filled in 1948. Queen's Creek Dam was built on the alluvial material filling the creek bed. The seal trench was not excavated to rock and only a limited amount of foundation grouting was done. All the water passing the dam is believed to be leakage through the foundation. It amounts to 23 gallons per minute with a full reservoir. It has not increased since the reservoir was first filled.

Queen's Creek Dam has a rather thick impervious core made of relatively poor material and only a thin rock blanket upstream from the core. The upstream slope of the dam is 2. horizontal on 1. vertical. The dam was in part an experiment to determine the effect of an alluvial foundation and a steep upstream slope. The small reservoir has been completely emptied on several occasions as rapidly as it was feasible to do so. No sloughing of the earth core has occurred under conditions of rapid drawdown. The settlement of the dam during nine years of operation has been 0.28 feet vertical and 0.18 feet horizontal. There has been no differential settlement.

The spillway consists of an unlined rock channel through the left abutment ending in a cascade a short distance below the toe of the dam. This spillway channel has a capacity of 14,000 c.f.s. and it is closed by two fuse plugs which will wash out if overtopped. A conduit under the dam takes water to the pipe line leading to the turbine and also to a valve having a flood discharge capacity of 100 c.f.s. The flood discharge conduit has been used several times. The spillway has not been used.

The only operation required by the Queen's Creek Dam has been to open the flood discharge valve during very heavy rains.

The only repair work necessary on Queen's Creek Dam was performed on the concrete conduit just above the flood discharge valve where a crack developed. This crack was sealed and the conduit is now operating satisfactorily.

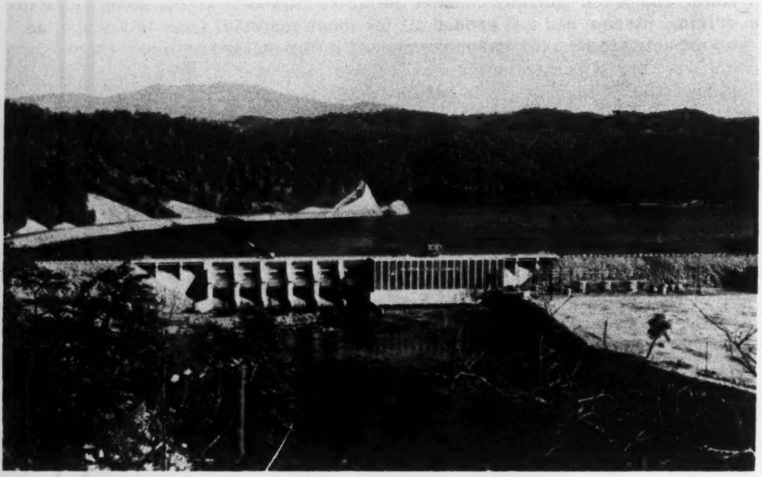


FIG. 13.—CHILHOWEE DAM, GENERAL VIEW

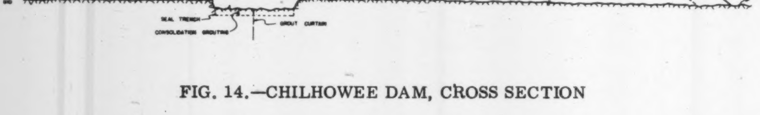

130'
21.5'
CREST ELEV. 883.0
MAX NORMAL POOL ELEV. 874.0
870
860
850
840
FILTER
FILTER
2.2
1.7
1.3
1.3
1.3
9'
15'
QUARRY ROCK
COMPACTED IMPERVIOUS EARTH
MAIN ROCK FILL
SMALL
MEDIUM
LARGE
COFFER DAM
SEAL TRENCH
CONSOLIDATION GROUTING
GROUT CURTAIN
10 0 10 20 30 40

FIG. 14.—CHILHOWEE DAM, CROSS SECTION

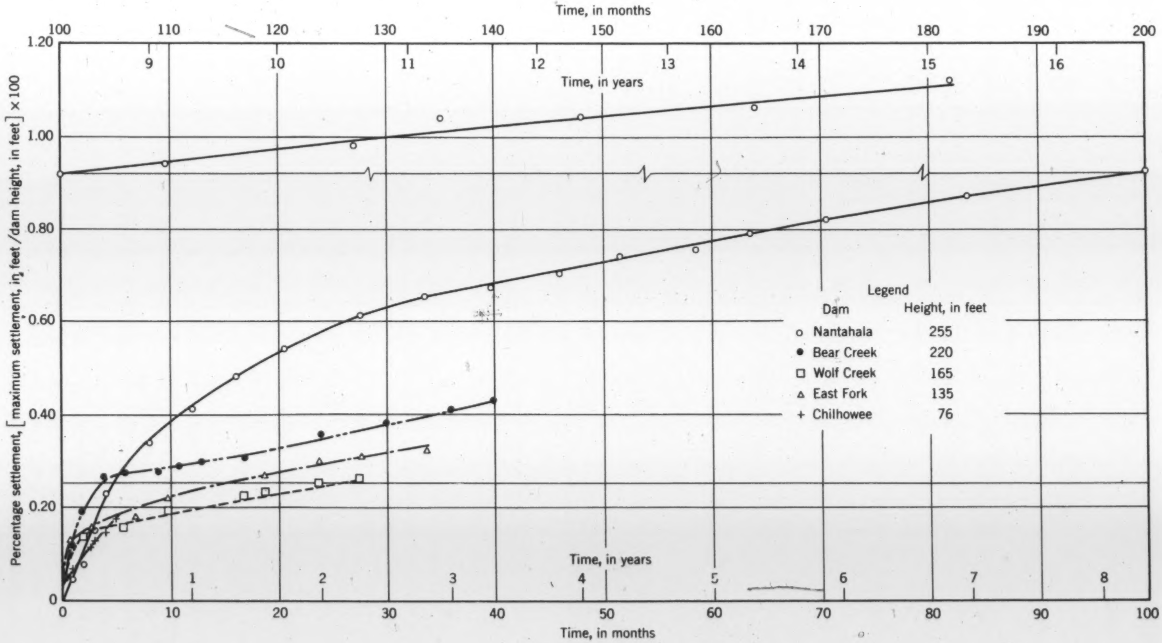


FIG. 15.—ROCK-FILL DAMS, SETTLEMENT CURVES

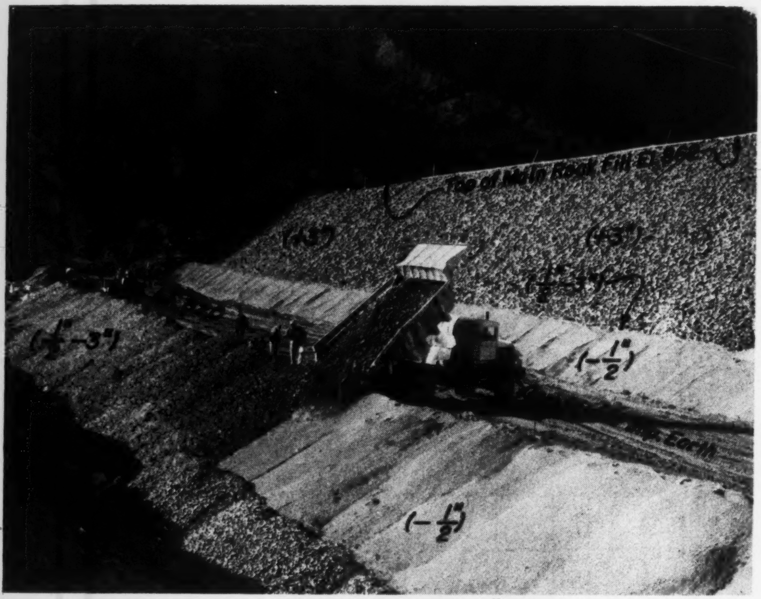


FIG. 16.—CHILHOWEE DAM, PLACING FILTER MATERIAL

No other maintenance has been required and the only maintenance which can be foreseen is replacing the fuse plugs if and when they are washed out by especially large floods.

Cedar Cliff

Completed August, 1952. Reservoir filled shortly thereafter. The foundation and abutments of the Cedar Cliff Dam appeared to be quite tight and only a small amount of grouting in the seal trench was done to reduce construction costs. The leakage measured immediately below the dam was 32 gallons per minute with the reservoir filled and in dry weather. It has not increased since the reservoir was filled. It is believed that all of the measured leakage comes through the foundation.

Soon after Cedar Cliff Dam was completed, a crack developed along the crest between the upstream rock blanket and the impervious core. The reservoir was later completely emptied at a rapid rate and the entire dam examined for sloughing or for areas of distress. No areas of sloughing or distress were found to exist. The crack is believed to be a result of not sluicing the upstream rock blanket and thereby not obtaining rock to rock contact throughout the blanket. The dam performance to date has been completely satisfactory. Settlement during five and one-half years of operation has been 1.08 feet vertical and 0.26 feet horizontal.

Several floods have passed through the gated spillway channel and over the cascade to the river channel close to the downstream toe of the dam. The cascade has operated perfectly to absorb the energy of the falling water with some small enlargement of the area at the bottom of the cascade. No flood in excess of the gate capacity has so far occurred and the fuse plugs have not been washed out.

The flood gate is automatically float controlled. Operation has been limited to the discharge of trash through the flood gates.

The dam has not required any repairs.

No maintenance has so far been required and none is foreseen except the replacement of a section of fuse plugs if it should be washed out.

Bear Creek

Completed in October, 1953. Reservoir filled some three months later.

Only a limited amount of grouting was done in the seal trench. Leakage past the dam with the reservoir filled and dry weather has been consistently 60 gallons per minute. It is believed that all the measured water passes through the foundation. The rate of leakage has not changed since the reservoir was filled.

The Bear Creek Dam was placed in one high lift except for the top 40 feet. The main rock fill was heavily sluiced but the upstream rock blanket was not sluiced at all. As at Cedar Cliff a crack developed along the crest between the upstream rock blanket and the impervious earth core. Experience at Cedar Cliff and the performance of Bear Creek Dam have shown that this crack in no way affects the operation of the dam. The maximum settlement has been 0.95 feet vertical and 0.54 feet horizontal. The reservoir has been drawn down on several occasions but has normally operated within a few feet of full.

The Bear Creek spillway is an unlined channel in the rock of the right abutment. It is controlled by one float operated automatic Tainter Gate with a "Hundred Year Flood" capacity and a series of fuse plugs with a spillway design flood capacity. It terminates in a series of three cascades cut in the rock which returns the flood water to the river bed. The spillway has been used several times; once for a considerable period with a maximum discharge of 3,800 c.f.s. There has been no erosion of the spillway channel and no enlargement of the cascades. The fuse plugs have so far remained in place.

The only operation required at this dam is the discharge of trash through the spillway gate.

No repairs have been required.

No maintenance has been required and none is foreseen in the future except painting the spillway gate and replacing the section of fuse plug if it should be washed out.

Wolf Creek Dam

Completed in May, 1955—Reservoir filled shortly thereafter.

The foundation at Wolf Creek is relatively less watertight than the foundation of the dams previously constructed. Considerable grouting was required to limit the loss of water. The leakage measured below the dam with the reservoir full in dry weather is 110 gallons per minute. This relatively large leakage could have been reduced by further grouting in the seal trench. The leakage does not endanger the dam structurally and the cost of the grouting required to substantially reduce it would have been more than the value of the water saved.

Settlement of the Wolf Creek Dam since the reservoir was filled has been 0.44 feet vertical and 0.28 feet horizontal. The settlement varies uniformly from the highest section of the dam to the abutments. There has been no differential settlement.

The Wolf Creek spillway is an unlined channel cut in the rock of the right abutment. The flow in the spillway is controlled by one float operated Tainter Gate and two fuse plugs. The channel ends in two cascades which returns the water to the stream bed not far below the toe of the dam. Several minor floods have passed through the spillway with no erosion and no damage. The fuse plugs have remained intact.

The dam has not required any operation.

No repairs have been required.

No maintenance has been required. The only foreseeable maintenance is painting the Tainter gate and replacing the fuse plug if and when it is washed out.

East Fork

Completed in May, 1955. The reservoir was filled shortly thereafter.

The rock foundation for the East Fork Dam appeared relatively tight and only a moderate amount of grouting was done in the seal trench. The leakage past the East Fork Dam with the reservoir filled in dry weather is 20 gallons per minute all of which is believed to be through the foundation.

The settlement of the East Fork Dam has been 0.43 feet vertical and 0.16 feet horizontal diminishing uniformly from the highest section of the dam to the abutments. There has been no differential settlement.

The East Fork spillway is an unlined channel in the rock of the right abutment. The flow in the spillway is controlled by a float operated automatic Tainter Gate and two fuse plugs. The spillway terminates in a cascade. Several small floods have passed through the spillway without damage. The fuse plugs have remained intact.

No operation has been required at the dam. Some badly cracked rock in the right abutment has been grouted after the dam was placed in operation.

No maintenance has been required and none can be foreseen except painting the Tainter Gate and replacing a section of fuse plug if it should be washed out.

Chilhowee

Completed in August, 1957. Reservoir filled shortly thereafter.

The foundation of the rockfill section of Chilhowee Dam appeared to be quite tight. The seal trench was thoroughly grouted. No leakage has been measured or observed since the dam was placed in operation.

Settlement of the rockfill section of Chilhowee Dam has been 0.16 feet vertical and 0.12 feet horizontal. There has been no differential settlement.

All the flood waters are passed through the concrete spillway section of the dam and do not affect the rockfill section.

The rockfill section requires no operation.

The rockfill section has not required any repairs and no maintenance. No repairs or maintenance can be foreseen in the future.

CONCLUSIONS

All of the rockfill dams leak but only a very small amount. The leakage does no harm to the structure and its economic value is negligible. It is believed that none of the measured leakage passes through the earth core of any of the dams.

All rockfill dams settle. Settlement of Alcoa's rockfill dams has been less than allowed for in the design. The rate of settlement decreases with time so that the settlement will shortly become too small to measure. (Fig. 15)

While the service life of these dams has been short, the spillways have handled all the floods without damage. The spillways are believed to be ample to pass any flood which can reasonably be expected to occur without damage to themselves or to the dam. The use of one or more cascades to return the water from the spillway channel to the stream bed is very effective in limiting the amount of material moved by the flood water.

The dams themselves have required no operation procedures except the handling of trash through the sluice gates. The dams have required no repairs or maintenance nor can any be foreseen in the future except the maintenance required by the sluice gates and the replacements of a fuse plug if it should be washed out.

One other important factor of performance may well be mentioned. The cost of Alcoa's rockfill dams has been much less than that of any other type of structure for these sites. The resulting low annual cost has been very satisfactory.

The sloping core rockfill dams so far designed and constructed by Alcoa have performed to the entire satisfaction of their owners and operators.

ACKNOWLEDGMENTS

The Power Engineering Division of the Aluminum Company of America, with the cooperation of Mr. B. J. Fletcher, Vice President, General Manager of Engineering has furnished the information contained herein.

DISCUSSION

F. L. LAWTON,[2] F. ASCE.--The author's use of fuse-plug fills in connection with several developments embodying rockfill dams has no doubt contributed materially to the low development costs achieved. Could the author indicate the criteria used in the design of the fuse plugs? By how much must a fuse plug be over-topped before it will wash out?

---

[2] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

The experience at Cedar Cliff and Bear Creek Dams associated with the development of a crack along the crest between the upstream rock blanket and the impervious core is interesting. Would the author care to indicate whether, in his opinion, this is an indication of slippage of the upstream rock blanket on the underlying filter zone or of the filter zone on the impervious core? It is believed these two are the only known instances of such an occurrence although they have been reported for a number of rockfill dams with a central core of earth supported by heavy rock shoulders.

The low capital and annual costs noted for the rockfill dams built under the author's supervision is of considerable importance in relatively inaccessible locations.

JAMES P. GROWDON,[3] **F. ASCE.**—It is a pleasure to report, in reply to Mr. Lawton, that there has been no lateral movement of the fill on the abutments toward the maximum section on any of the dams referred to by the writer.

Fuse plugs for the control of seldom-used spillways should have a pilot channel which will wash out when overtopped by 1 ft (more or less) of water. The rest of the fuse plug should never be overtopped at all; it should wash out laterally at a rate predetermined by the designer and constructed of materials which will meet the designer's specifications.

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[3] Consultant, Aluminum Co. of America, Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3071

ROCKFILL DAMS: KENNEY AND CHEAKAMUS DAMS

By William G. Huber,[1] F. ASCE

With Discussion by Messrs. J. Barry Cooke; F. L. Lawton; and William G. Huber

SYNOPSIS

Site and materials data, design criteria, construction procedures and some performance records are presented in this paper on two zoned rock fill dams in British Columbia. One is founded on rock and the second on a mud slide.

INTRODUCTION

Fill dams are the natural choice for first consideration by hydraulic engineers in British Columbia when required to design an impounding structure and of the several types, rockfills are preferred because rock is almost always available in ample quantities, whereas the fine grained sediments required for an earth or even thick core dam are quite often in short supply. The geology of the stream valleys in particular is featured by the effects of repeated, long term and intense glaciation, which now appear as extensive filling or plugging of the ancient channels by drift and till. With few exceptions the modern all-rock dam sites are associated with plugs so high and resistant as to divert the streams permanently over rock through which they have eroded canyons in post glacial times. Kenney Dam site is of this type, while Cheakamus Dam rests on unconsolidated valley fills.

Note.—Published essentially as printed here, in June, 1958, in the Journal of the Power Division, as Proceedings Paper 1671. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The criteria for a rock fill dam appear to be amply satisfied by Kenney, but Cheakamus calls into play definitions of the term.

The editors of Engineering News Record in the issue of October 24, 1957 proposed to define a rock fill dam as one made up of more than fifty percent rock. However, Mr. A. L. Alin, by letter appearing in that publication November 28, 1957, suggested the following: "A rock fill dam is a type of dam which, for its stability, depends on loosely dumped rock." It appears that any definition must be broad and relatively loose - and should establish the nature of a rock, as well as the manner of use and the purpose served. A fill of coarse gravel would be built in the same manner and have all the qualities and characteristics of a dumped rock fill but should probably not be designated in those terms. If Mr. Alin's definition is narrowed by adding the words ". . . of quarry shapes and sizes" then it is both convenient and accurate to classify Cheakamus as a rock fill dam.

Vogues or schools so evident in the history of the engineering construction art also manifested themselves in British Columbia, except that dam construction on a larger scale began there when concrete was a preferred medium. Consequently the current success of rock fills in that area is entirely a post war development, and based on the following economic factors:

1. Utilization of more remote sites for storage and power development which involve longer supply lines and transport costs as elements of total construction cost. With rock available locally, capital cost becomes less than for an equivalent concrete dam.

2. Great improvements in quarrying equipment and techniques, and the steady growth of heavy excavation and haulage units.

3. Practical elimination of maintenance expense during operating life of the dam, which is an increasingly important element of cost, generally accentuated in mountainous locations by difficulties of access and more severe weathering action by high altitudes.

4. In many cases, rock-fill dams can be built on the natural river bed materials so often found at topographically favourable dam sites, because high velocity stream flows have carred away the fines and reworked the coarser particles into the equivalent of a first quality dumped rock fill.

In addition to the several cost advantages offered by rock fill construction in certain locations, these embankments provide desirable inherent toughness and reliability. They have the capacity of self healing under shock as from an earthquake or bombing attack, and are able to pass a large amount of leakage without danger of failure or even permanent damage. This latter feature of dumped rock embankments suggests the concept of rock fill dams as the ultimate development of rock or gravel toe drains commonly incorporated in earth dams.

Performance of Kenney Dam

Kenney Dam, located in the central part of British Columbia on the Nechako River and containing a total of about four million cubic yards of all materials, was installed to impound about twenty million acre feet of water for the Kemano power plant of the Aluminum Company of Canada (Alcan) B. C. Development. Reservoir flood spillway and power intake are remote from the dam which has a hydraulic height of about 275 feet to top reservoir elevation 2800, and a total height of 325 feet above the low point of the foundation.

The section is of the sloping core type with finished upstream slope of 2.5:1 and average downstream slope of 1.75:1. Width of crest is 40 feet and minimum freeboard above full reservoir is 20 feet. Length of the dam at the top is approximately 1600 feet, arched in an upstream direction to a radius of 5729 feet, (1° curve).

Site

The damsite is in a post glacial channel eroded by the Nechako River through a series of basalt flows dipping about 7° in a downstream direction and forming the upper end of the Grand Canyon of the Nechako. At this point the river which carried discharges ranging between 1500 and 30,000 cusecs, flowed in a steep walled canyon about 80 feet deep on a gradient approximating 3%. Above the rim of this rock canyon the sides of the valley were burdened with varying depths of glacial till eroded to slopes of about 35°.

Design

The adopted design closely followed that developed for the Nantahala Dam of the Aluminum Company of America, except for the flattening of the downstream slope to add safety in the event of earthquake shocks. The general principles of this design are to provide a supporting member of dumped rock, a graded filter on the upstream slope of the rock section, a rolled fill sloping core, a superimposed filter of sand protected in turn by pitrun gravel and an upstream surface layer of quarry run rock to absorb wave action. At Kenney, the upstream cofferdam was incorporated in the blanket covering the rolled fill core. Abutment contacts downstream of the blanket were everywhere stripped to rock, but the boulder-gravel deposit in the original river bottom downstream of the core and filter contacts was accepted as foundation for the dumped rockfill. Plan and typical section are shown on Fig. 1.

Construction Materials

Rock fill was obtained from strata of massive basalt in the left abutment, also most of the 3" - 10" downstream filter zone was produced by crushing and screening reject from the quarry. Scattered deposits of alluvial origin and mostly on the right bank supplied all sands, some filter gravel by screening, and the pitrun fill overlying the core. These natural sands and gravels were poorly graded and in relatively small pockets which led to a regular

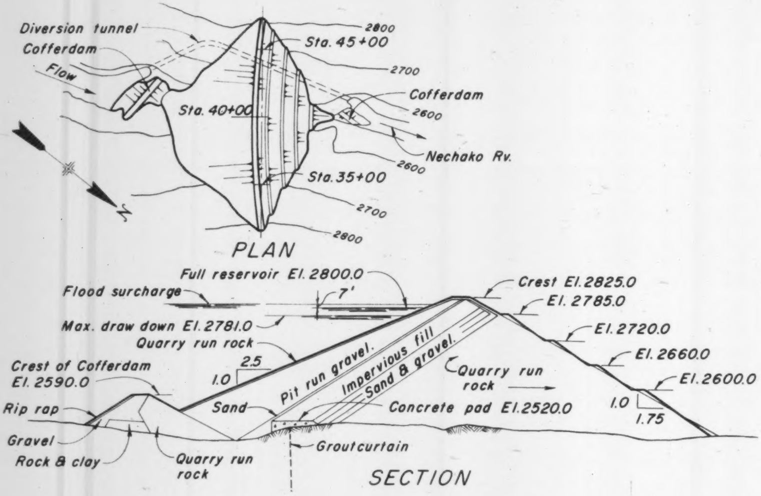
Diversion tunnel
Cofferdam
Flow
2800
Sta. 45+00
2700
Cofferdam
2600
Sta. 40+00
Nechako Rv.
2600
Sta. 35+00
N
2700
2800
PLAN
Full reservoir El. 2800.0
Flood surcharge
7'
Crest El. 2825.0
El. 2785.0
Max. draw down El. 2781.0
Quarry run rock
El. 2720.0
El. 2660.0
2.5
1.0
Pit run gravel.
Impervious fill
Sand & gravel.
Quarry run rock
El. 2600.0
Crest of Cofferdam
El. 2590.0
1.0
1.75
Rip rap
Sand
Concrete pad El. 2520.0
Gravel
Groutcurtain
Rock & clay
Quarry run rock
SECTION

FIG. 1.—KENNEY DAM

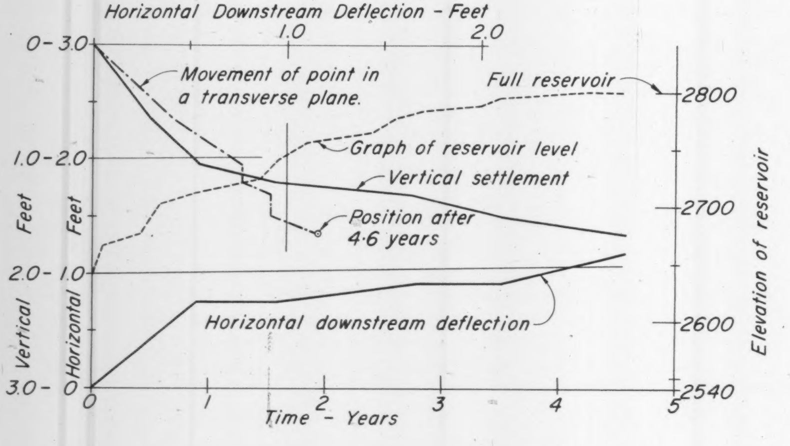

FIG. 2.—PERFORMANCE OF KENNEY DAM

program of exploration and pit development in order to maintain production, and eventually the haul distance from pits was in excess of two miles. Impervious fill material was borrowed from a large deposit of boulder till on the right bank and about a mile downstream from the dam axis. As explored by a pattern of augur holes 20 feet in depth and several test pits, this till average approximately 10% clay sizes and contained about 8% of water.

Construction

The Nechako River was diverted around the left abutment through a 32 foot horseshoe tunnel about 1500 feet long, by an upstream cofferdam approximately 90 feet high. When unwatered, the river bed proved to be a maze of rock pinnacles, deep holes, and scour channels. The core contact area on the abutments was trimmed to slopes not steeper than 1.5:1, gunited to seal the surface of the vesicular lava and broken basalt, and a grout cutoff was constructed by drilling through the gunite seal coat. At some points, concrete was used to build out re-entrant steps in the core contact area. The gunite seal coat proved capable of withstanding shallow consolidation grouting pressures of about 30 p.s.i. and forced some horizontal distribution of the grout.

A foundation grout curtain located on the center line of the core contact area was completed by drilling about 75,000 linear feet of holes which took a total of 48,000 bags of cement at pressures up to 200 p.s.i. Depths of holes in the sequence were respectively 30', 75' and 150'. Downstream of the core contact, the abutment rock was trimmed to 1:1 slope as far as the axis of the dam, thence warped back to natural slopes within the limits of rock fill contact.

A base for the impervious core was constructed of concrete, filling in the extremely irregular rock surface of the river bed and finished to a level surface about 80' wide and 100' long at El. 2520 on which the first of the clay fill was compacted.

To produce rock fill material, a face about 1500 feet long was developed in benches by coyote hole quarrying, requiring slightly less than an average of one pound of blasting powder per cubic yard broken. Despite very favourable cores from exploratory drilling of the basalt deposits, an unexpected amount of residual clay and fines were produced by each blast, which required selective loading of the product in the quarry and constant attention to allocation of the loads. When the fines content of a load exceeded 15% by volume, it was dumped down a high face, with resultant segregation of part of the rock content which was then reclaimed along the toe of this slope. Other loads considered unsuitable for the main fill were put through a crushing plant and screened to produce the coarser filter sizes. All except the upper portion of the rockfill was built out from the left abutment in lifts approximately 40 feet high, with each load sluiced down the slope from the top of the lift by skid mounted monitors delivering about two parts of water to one part of rock. Most of the sluicing water was reclaimed at large sumps just inside the downstream cofferdam and pumped back through the monitors.

Filters

The three filter zones downstream of the core were placed by truck in 10" to 12" lifts, watered and compacted by caterpillar tread tractors except that the sand zone for 100' from each abutment was spread in thinner layers and given extra compaction. The zones were brought up in steps, each about five feet higher than the adjacent upstream zone. Two patches of a fourth filter of 1/4" - 3/4" sizes were placed above the principal break in the rock abutments as trimmed, to increase the safety factor during settlement of the completed embankment. A large portion of the screened and stockpiled sand for the fine filter zone contained so much silt that when moistened in layers on the dam, it became somewhat plastic and was insufficiently free draining; consequently that fraction which contained more than about 2% of silt had to be washed before hauling to the embankment.

Impervious

The boulder clay used for the sloping core or impervious diaphragm was first broken up with rippers to about 30" depth, then moistened in the pit by a dozer equipped with raker teeth and hauling a tank wagon, feeding water to the point of each rake tooth. This irrigation plus dozer harrowing distributed the water through the loosened till, which was then allowed to temper for several days before loading into pushed scrapers for transport to the dam, where it was placed in 10 inch lifts, and compacted by 16 passes of sheepsfoot rollers to 95% of Standard Proctor density. Despite the processing in the pit, some hand picking of oversize (6") rock was required after spreading on the embankment. Except at contact with the abutments, the core was placed on the dry side of optimum, that is, about 12% moisture by weight, which gave a compacted dry weight averaging 110 lbs, per cu. ft. At base and abutment contacts water content was somewhat increased, rocks larger than 2" were removed, and compaction was accomplished by pneumatic tampers.

The pitrun sand and gravel overlying the upper sand filter was placed with inter-bedded pervious zones to assure freer drainage under drawdown conditions. It should be noted, however, that drawdown from full reservoir level will probably never exceed 20 feet.

The dam proper was built in the relatively few months between May and November 1952, at peak rates of 50,000 cu. yds. place measure of all classes per day of two shifts. Reference is made to H. Jomini's article in the Engineering Journal, November 1954 for additional details of construction.

Two sets of strain measurement points were embedded about 4.0' below the surface of the fill and at spacings of 100' along the base line. The series of 13 monuments near the top of the core initially were on a vertical camber or curve of 3.5' middle ordinate. The second set of 11 monuments along the upper berm of the rock fill were initially in a horizontal plane at el.2782.75 or about 36' below the crest monuments.

Performance

The time rate hydrostatic loading of Kenney Dam is shown in Fig. 2 starting October 8, 1952 and reaching full reservoir elevation 2800 early in 1957. Although the site presented the possibility of leakage through the abutments along contacts between successive lava flows beyond the limits of the foundation grout curtain, no seepage has been detected coming from the abutments or through the embankment proper. The vertical settlement and horizontal movement both downstream and longitudinally at each of the two sets of monuments have been measured six times since December 1952, most recently in July 1957. The direction and magnitude of these deflections or movements are illustrated by Fig. 2, representing data obtained through the courtesy of Power Operations Division of Alcan, B. C.

The graphs were plotted using only the vertical and downstream displacements of crest monuments #6, 7 and 8, recording for the central 300' of dam length and averaged to smooth out any effects of probable errors in surveying and possible other localized movements. After the first eleven months, the approach of the graphs to the horizontal is quite noticeable but an increased divergence was reported for the year 1956 to 1957. Theoretically, this angle should decrease to near zero in another decade. The largest vertical settlements, which have occurred near the maximum section of the dam, are approximately 1.5 feet or 30% of the vertical camber provided at that point.

While the vertical and downstream or offset movements conform to expectations, the observed lateral or cross-valley displacements of the plates are inconsistent with theory, even after making all reasonable allowance for possible errors of measurement. The center point of a fill in a symmetrical valley should hold its original cross-valley position with reference to fixed points at the ends of the dam, during vertical and downstream displacements, likewise intermediate points if compressive or tensile strains are uniformly distributed. At Kenney Dam both on the crest and along the berm, the total lateral movement to date approximates 6" at the center reducing to about 2" near the left abutment, and consistently towards that end of the dam.

Consequently, a particle has moved vertically downwards, horizontally downstream, and laterally towards the left abutment, hence the resultant path could be represented by the diagonal of a parallelopiped.

No maintenance work of any kind has been required to date and none is likely to be necessary in the foreseeable future.

Cheakamus Dam

The Cheakamus Dam completed in 1957 is located on the Cheakamus River, about 25 miles north of the head of Howe Sound in British Columbia, as part of the 190,000 HP, 1125' head Cheakamus Development of B. C. Electric Company Ltd. The structure has a maximum height of 88 feet above foundation and will impound about 40,000 acre feet for release through a tunnel intake nearly one mile distant. The right abutment of the embankment is a combination gravity concrete diversion and spillway structure for discharging about 45,000 cusecs maximum.

Site

The normally complex geology of the area is in this instance further complicated by a large mud or landslide which originated on a flank of Mount Garibaldi about 100 years ago and stopped as a flattened cone or fan damming the Cheakamus River to form Stillwater Lake. The conditions imposed by this event became the dominant feature in the engineering and construction of Cheakamus Dam.

The slide followed the valley of Rubble Creek which was then probably in a rock canyon, and overfilled the alluvial and bedrock channel of the post glacial Cheakamus River. The foundation of the dam, therefore, in succession upwards is an extremely broken and irregular granite base, followed by glacial and alluvial sand and gravel overlain by varying depths of the landslide material which is hereafter called Rubble Creek Wash. In the eastern portion of the valley basalt is in contact with the granite base or with mantle on the granite.

Drill holes showed wash material in excess of 200' thickness. The slide broke off large trees of the living forest, enveloped the stumps and trunks and swept the tangle of loose forest growth towards the terminal contacts with the western slopes of the valley. Since the date of the slide the river had cut down into the wash about 35' and lowered the lake level.

Matters of concern in the investigation of the site included the following possibilities:

1) Under seepage through talus along the rock walls of the earlier valley.
2) Under seepage through the alluvium beneath the wash.
3) Under seepage through zones between the granite and superposed basalt.
4) Under seepage through fractures in the granite.
5) Under seepage through porous buried forest between the wash and alluvium.
6) Deficiency of strength and watertightness of the Rubble Creek Wash itself.

In all, nearly 100 exploratory holes were drilled to trace out the patterns of the foundation and test pits were dug in the wash, one to a depth of 40 feet.

Design

One of the earlier axes investigated which incorporated an outcrop of basalt surrounded by wash was eliminated because of probable large talus accumulations imperfectly sealed by landslide material. The site adopted is just upstream of the section where the river had cut to the rock forming the indicated right wall of the pre-slide channel. This placed the entire embankment on Rubble Creek Wash except at the right abutment contact with granite and the channel portion of the rock fill. In succession from the right end to the left end of the dam axis the drill holes showed thicknesses of Rubble Creek Wash to be 0', 62', 15', 35', 54', 122', and 126'.

This location utilized the upstream area of the slide fan as a blanket, eliminated a saddle dam, simplified stream diversion and reduced the overall length of structures. As site investigation progressed and the ground was opened up for construction, questions of water loss from the reservoir were resolved favorably. Several trenches were cut along the contact of the wash with the right or west wall rock of the valley, and filled with water. The losses were so small as to obviate special sealing treatment. Along the left side of the reservoir, the wash extended over possible talus up to full reservoir level, and sufficiently lengthened the path for seepage from farther upstream.

Adequacy of cutoff on this side was assured also by the great depth of consolidated wash in the pre-slide river canyon and extending well up the Rubble Creek Valley. Similarly, any seepage paths through the alluvium were considered to be sufficiently lengthened by this heavy blanket, which was found to have settled into tight contact with standing tree trunks. Grouting and water tests showed that jointing in the granite would introduce no hazard.

Since the slide material would be weak and unstable when again saturated, flat slopes were selected, and the upstream toe in the channel was buttressed by additional fill and the cofferdam.

The design of the dam includes a downstream supporting element of dumped rock with the center of this rockfill bearing on a ledge exposed in the natural streambed. Upstream of the rockfill and three filter zones, the impervious member is made up of Rubble Creek Wash founded on the same material as present in the streambed and left abutment, and on the old granite of the right abutment. This section was finished to a 4:1 slope down to el. 1190 then carried level about 200 feet to contact with the upstream cofferdam. Depth of the slide material at the upstream end of the impervious section is in excess of 200 feet. The dam was overbuilt to a crest camber of 2' maximum as allowance for settlement.

The rock section was given an average downstream slope of 1.6:1 and is underlain by sand and gravel filters and a system of tile drains. The filters provide against the migration of fines from foundation wash or along contacts between wash and ledge rock. Seepage collected in the system of under drains is trapped behind a small impervious dike at the toe of the maximum rock section and discharged through a gauge house equipped with measuring weir. Because of the possibility of leaks through the shattered granite below the mudslide material and earlier overburden, permanent drainage wells were installed at two points for observation and measurement. A plan view and maximum section are shown as Figure 3.

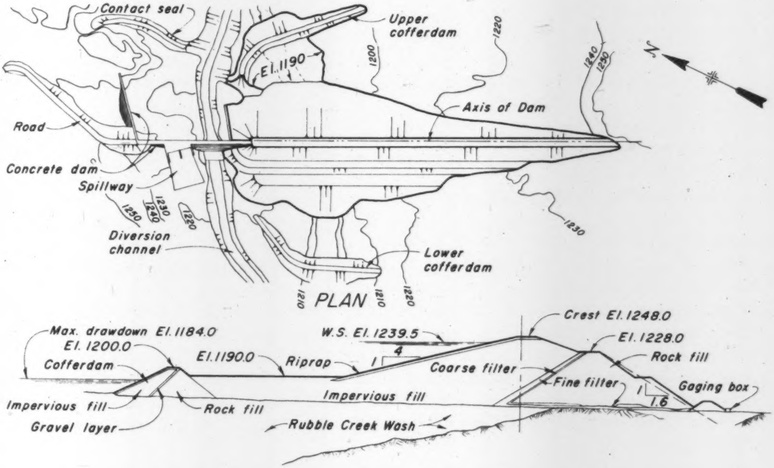
Contact seal
Upper cofferdam
1220
1200
El. 1190
1240
1250
N
Axis of Dam
Road
Concrete dam
Spillway
1230
1240
1250
1220
1230
Diversion channel
Lower cofferdam
1210
PLAN
1210
1220
Max. drawdown El. 1184.0
El. 1200.0
Cofferdam
El. 1190.0
Riprap
W.S. El. 1239.5
1
4
Crest El. 1248.0
El. 1228.0
Rock fill
Coarse filter
Fine filter
1
1.6
Gaging box
Impervious fill
Impervious fill
Rock fill
Gravel layer
Rubble Creek Wash

FIG. 3.—CHEAKAMUS DAM

Construction Materials

Rock fill was obtained from stripping the river bed, excavations for diversion channel and concrete structures, oversize rock from grizzly screening of Rubble Creek Wash and from talus accumulations along the valley wall. Sand and gravel for filter layers was produced by screening river deposits, while the "quarry run" or 4'' maximum size rock for blanket was selected from spoil piles at the site or at the east portal of the power tunnel. The Rubble Creek Wash material for the impervious zones was shovel excavated and trucked from pits near the south end of the dam, which yielded a typical slide mixture of sand, silts, a very small amount of clay and many small to large sub-angular rocks representative of the Garibaldi Mountain extrusives. The natural water content of this mixture was just above optimum and the permeability coefficient is 10^{-3} to 10^{-4} cm per second.

Construction

After excavation of a diversion channel through the rock of the right abutment and completion of an upstream cofferdam of rock and Rubble Creek Wash, the foundation area of the dam was prepared by stripping the surface to fresh unweathered wash where this material would become the foundation, removal of loosened rocks and exposed concentrations of rock and compaction by loaded scrapers. Rock along the right abutment and in the river bed was stripped clean and areas to be in contact with the impervious zone were gunited to fillet the re-entrant corners and otherwise smooth the surface. In addition a curtain of pressure grouting was extended from the gravity concrete sections of the dam upstream parallel to the diversion channel cut.

Springs in the foundation area of the impervious zone were piped to sumps - extended upwards with the embankment and finally backfilled with selected soil through which the pipes were plugged by pressure grouting. Similar springs in the rock fill area were confined by dikes and piped permanently by a tile line in graded filter to a gage box at the downstream toe.

In the area of the dam, contact zones between bedrock and wash were cleaned out by trenching and sealed with a layer of filter capped by selected Rubble Creek Wash.

After placing drain pipes, spreading and compacting a 3' layer of fine filter followed by 2' of corase filter and a thin lift of compacted small rock, the dumped rockfill was brought up in approximately 20 foot lifts, timed to keep ahead of quarry run, coarse, and fine filters in that order upstream towards the impervious zone. The filter between the dumped rock and wash of the left river bank was made up of 15' of fine filter blanketed by 3' of quarry run. Since the sloping filter zones were too thin to be placed by dumping from trucks at grade, the contractor generally utilized a high lift loader or small dragline to build them by rehandling stocks of the material delivered by truck from the top of the rock fill. Fine filter sizes ranged between #40 and 1'' and coarse filter between 1/4'' and 1'' or 2''. No sluicing water was used on the dumped rock.

First lift in the impervious zone consisted of three 8-inch layers of Rubble Creek Wash which had been grizzled to remove stones above 6 inches in size. For compaction at contact with rock or concrete the pit material was similarly screened to exclude stones more than 4 inches in size. Above these foundation lifts the Rubble Creek Wash was taken directly from the pits, excluding rocks more than 18 inches in dimension, and compacted in 24 inch layers by hauling equipment. A loaded scraper was used to compact the foundation layers.

A large portion of the rip rap for the upstream slope was supplied by raking out oversize rock from the Rubble Creek Wash as it was spread on the embankment. Due to the rock content of the material, an equivalent armoring would have been formed by wave sorting of the mixture during several operating seasons.

The reservoir created by Cheakamus Dam reached el. 1240 in October 1957, about five weeks after the diversion channel was plugged. Seepage measured at the gauge house is practically constant at 0.10 cusecs, except for increases due to rain seeping through the rock fill to filter zones, and the structure to date appears much less pervious than expected when the project was undertaken three years ago.

SUMMARY

The requirements for storage dams at two remote and relatively inaccessible sites in British Columbia have been met by zoned embankments of local materials. Although the Kenney site was suitable for a concrete gravity dam, the cost would have exceeded that for rockfill by about 50%. Masonry construction was not even considered for Cheakamus Dam where the special and rather complex foundation conditions could be satisfied only by a relatively flexible, self-healing blanket design.

ACKNOWLEDGMENTS

Detail design of Kenney Dam was by British Columbia International Company Limited. Consultants were Doctors Karl Terzaghi and J. P. Growdon and Mr. I. C. Steel, and construction by Mannix Limited, with soils engineering by Dr. R. C. Hardy and Ripley & Associates.

Design and construction supervision of Cheakamus Dam was by B. C. Engineering Company Limited under Dr. T. Ingledow, President and Dr. Karl Terzaghi, Consultant. Emil Anderson Construction Company built the structure and soils testing was done by Ripley & Associates.

DISCUSSION

J. BARRY COOKE,[2] **F. ASCE.**—The author has provided valuable comment and data on two particularly notable rockfill dams. Kenney Dam was a substantial step from Nantahala Dam and has an important place in the successful development of the sloping core rockfill dam. Cheakamus, though not particularly high, is of substantial significance in its being constructed on glacial, alluvial and talus deposits without cutoff to bedrock. The Cheakamus Dam and the Bersimus Dam are examples of careful adaptation of rockfill dams to difficult sites. The proposed High Aswan Dam, to be constructed on a several hundred foot depth of sand and gravel, will further lead to increased use of rockfill dams at difficult sites.

The author states that the lateral movement (cross-valley component) of settlement points on Kenney Dam seems inconsistent with theory and says, ". . . The center point of a fill in a symmetrical valley should hold its original cross-valley position with reference to fixed points at the ends of the dam, during vertical and downstream displacements . . . At Kenney Dam both on the crest and along the berm, the total lateral movement to date approximates 6 inches at the center reducing to about 2 inches near the left abutment, and consistently towards that end of the dam . . ." These movements indicate that lateral compression has developed in the left half of the dam and lateral tension in the right half, rather than the expected tensions near both abutments and compression at the center. More detailed data would be of value. The writer agrees that the center of a symmetrical rockfill dam should not move laterally if the dam were symmetrically constructed. The lateral movement is basically from each abutment toward the center, but it is also related to the direction of dumping. The rockfill at Kenney Dam was constructed from the left to right bank, and that is believed to be the reason for the lateral movement being from the right to left bank. Lateral movement is greatest in the direction opposite to that of dumping the rockfill. Measurements on several dams indicate this and reasoning can be developed to make it appear logical.

The lateral settlement at Salt Springs Dam is greatest in the direction opposite to that of dumping the rockfill. The lower 170 feet of Salt Springs Dam was dumped from right to left abutment. The next 75 foot lift was also, except for a narrow face lift from the left abutment. The top lift was dumped from both abutments but its influence on lateral movements is not very great. Fig. 4, of a paper[3] by I. C. Steele and the writer, shows that the compaction during dumping was greatest in the direction of the dumped slope, even when the settlement point was 100 to 150 feet back from the berm at which dumping was taking place. Points 3 - 5, of this Fig. 4, began to change direction of lateral movement as the fill berm progressed 200 to 300 feet beyond them. Fig. 8 of the same paper shows the later movement to be predominantly to the right. On Line F all points moved to the right, in the opposite direction from dumping of the major portion of the fill. Lateral movements of the 245 foot high Lower Bear River Dam No. 1 were small but the upper left abutment joint openings, Fig. 17A of that paper, indicate greatest lateral movement being opposite to direction of dumping the fill. All the rockfill of Lower Bear River Dam No. 1 was constructed from the right abutment.

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[2] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

[3] "Rockfill Dams: Salt Springs and Lower Bear Concrete Face Dams," by I. C. Steele and J. B. Cooke, Transactions, ASCE, Vol. 125, Part II, 1960.

The Paradela concrete face rockfill dam, 368 feet high, was all constructed from the left to right abutment and lateral movement was predominantly from right to left. Vertical joints near the right abutment opened one to several inches, whereas the vertical joints near the left abutment did not move or closed about one-half inch. The design and construction of the dam has been comprehensively presented.[4] The data on lateral movement at Paradela is given with the permission of Mr. Gomes Fernandes, Hidro Electrica do Cavado.

The lateral movements of the above three high concrete face rockfill dams appear to be substantially influenced by the direction of dumping the rockfill. Though Kenney Dam is a sloping core type, it is considered that its settlement should be similar to that of a concrete face rockfill dam, and the data presented by the author seems to confirm this for all components of the crest settlement. This similarity in crest settlement does not exist for the central core type of rockfill dam, for which all movements are less and are much less influenced by the rockfill.

It seems logical that lateral settlement should tend to be in the opposite direction to the direction of dumping. As dumping takes place the surface slides and impact of large rocks sliding down the slope tend to compact the fill in that direction. Also a substantial contact of the rocks in a line down the dumped slope is necessary to stop the rocks as they fall. The line of rock contacts is not as well established in a direction approximately normal to the dumped slope. When the differential vertical and downstream displacements take place the lateral readjustment would reasonably tend to be in the direction of least compaction; i.e., the direction opposite to the direction of dumping the rockfill. It appears that the magnitude of the effect of the direction of dumping on lateral movement is as great as the magnitude of the basic lateral settlement from both abutments to the center of a symmetrically constructed dam.

For dams of less than 325 feet in height these lateral movements are probably not of consequence. However, for higher dams it may be that consideration should be given to require a certain minimum amount of dumping from each abutment in order to minimize lateral tensions. Though the ideal design would require the dumped fills to close at the center, conditions are usually such that it is practical and economic to construct more dumped rockfill from one abutment than the other. Consequently, design restrictions on construction should be kept to a minimum. Since the lower lifts have the greatest effect on the lateral movement, the design requirements on the direction of dumping might be limited to them.

There is much remaining to be learned about the behavior of high rockfills and the adapting of rockfill dams to difficult sites. The publishing of construction data and detailed measurements on the existing dams will make it possible to design and construct higher rockfill dams with maximum safety

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4. "Rockfill Dams: Paradela Concrete Face Dam," by Luis Henrique Gomer Fernandes, Edgard de Loiveira and Nuvo de Vasconcelas Porto, Transactions, ASCE, Vol. 125, Part II, 1960.

and economy. The cooperation of Mr. William G. Huber, The Aluminum Company of Canada and B. C. Electric Company in presenting data on Kenney and Cheakamus Dams has resulted in a valuable contribution to this Symposium.

F. L. LAWTON,[5] **F. ASCE.**—The author has given an unusually interesting comparison in his treatment of the Kenney and Cheakamus dams, both of the rockfill type but varying materially in design of cross-section and especially in foundation conditions, Kenney dam being founded on rock and Cheakamus dam on a 100-year-old landslide. This discussion is confined to the performance of the Kenney dam since its construction.

The author's Fig. 2 illustrates the rate at which hydrostatic loading on Kenney dam was imposed. Reservoir filling began October 8, 1952, with full reservoir at el. 2800 being reached August 1957.

As stated by the author, no seepage has been detected coming from the abutments or through the embankment proper, although the possibility of leakage through the abutments did exist, because of the contacts between successive lava flows beyond the limits of the foundation grouting.

During construction, two sets of settlement (and deflection) movement plates were installed. One of these sets was located in the top of the impervious core about 5 ft. below the crest, which consists of rockfill with a gravel topping arranged in the manner shown by the author's Fig. 1. The second set was located about 3 ft. below the surface of the berm at el. 2785, in the top of the sluiced rockfill. The first set comprised 13 monuments spaced at 100 feet intervals along the base line and on a vertical camber or curve of 3.5 feet middle ordinate. The second series of 11 monuments in the berm at el. 2785.0 was initially in a horizontal plane at el. 2782.75 or about 36 ft. below those in the crest.

Location and other details of the settlement plates in the crest of the dam are given by Table 1, with corresponding data for those in the berm by Table 2. Settlement observations for the crest and the berm have been made on seven occasions, with the last in July, 1958. Data is given by Tables 3 and 4.

It will be noted:

1. The present settlement of both the crest and the berm monuments has been relatively uniform, percentage-wise. That is, the settlement is generally proportional to the height of the rockfill, at the monument.
2. Average berm settlement is about 42% greater than the average settlement of the crest.
3. Settlement of the crest has slowed down and appears to be virtually complete. (See Fig. 4).

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[5] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

4. Settlement of the berm has slowed down but does not appear to have reached the same degree of completion as that for the crest (See Fig. 5).

At the same time as settlement observations were made, measurements were made to determine the deflection movement of the monuments in a downstream direction, Table 5 giving essential details for the crest and Table 6 corresponding data for the berm. The great majority of the movements are in the downstream direction but a few measurements indicate an upstream movement. There appears to be some correlation between deflection and height of fill. The deflection expressed as a percentage of height of fill does not vary greatly, averaging about 0.36% for the crest and 0.47% for the berm. These fingers represent 67% and 61%, respectively, of the corresponding average settlements. The deflections appear to have reached a stable condition, with little if any further increase likely. In fact, there is an apparent decrease between July, 1957, and July, 1958, observations.

TABLE 1.—KENNEY DAM, CREST SETTLEMENT DATA

Settlement Plate No.	Location at chainage	Elevation of settlement plate Dec. 12-13, 1952	Elevation of foundation rock	Original height of settlement plate above foundation rock ft.
1	33 + 50	2816.34	2766.0	50.3
2	34 + 50	2817.42	2731.0	86.4
3	35 + 50	2817.89	2691.0	126.9
4	36 + 50	2818.94	2652.0	166.9
5	37 + 50	2819.43	2626.0	193.4
6	38 + 50	2819.69	2552.0	267.7
7	39 + 50	2819.82	2510.5	309.3
8	40 + 50	2819.73	2575.0	244.7
9	41 + 50	2819.44	2607.5	211.9
10	42 + 50	2816.96	2646.0	171.0
11	43 + 50	2818.26	2670.0	148.3
12	44 + 50	2817.40	2698.0	119.4
13	45 + 50	2816.30	2744.5	71.8

TABLE 2.—KENNEY DAM SETTLEMENT DATA, BERM AT EL. 2785

Settlement Plate No.	Location at chainage	Elevation of settlement plate on Dec. 12-13,1952	Elevation of foundation rock	Height of settlement plate above foundation rock,ft.
1	34 + 50	2782.82	2731.0	51.8
2	35 + 50	2782.79	2691.0	91.8
3	36 + 50	2782.76	2652.0	130.8
4	37 + 50	2782.71	2626.0	156.7
5	38 + 50	2782.69	2552.0	230.7
6	39 + 50	2782.70	2510.5	272.2
7	40 + 50	2782.70	2575.0	207.7
8	41 + 50	2782.70	2607.5	175.2
9	42 + 50	2782.78	2646.0	136.8
10	43 + 50	2782.78	2670.0	112.8
11	44 + 50	2786.95	2698.0	89.0

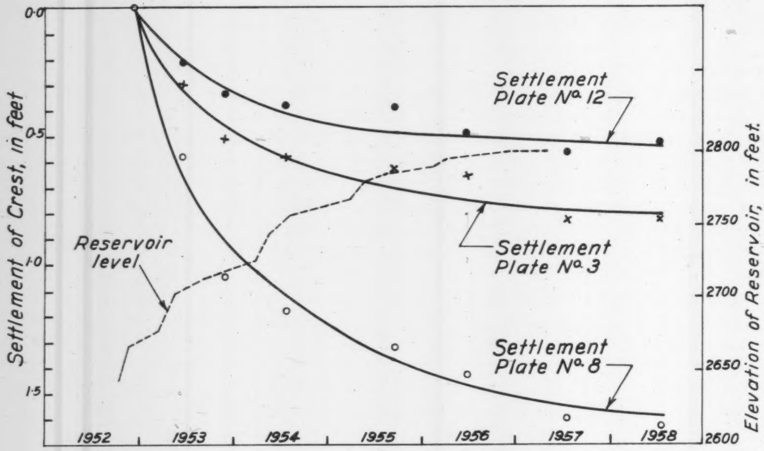
Settlement of Crest, in feet
0.0
0.5
1.0
1.5
Elevation of Reservoir, in feet.
2800
2750
2700
2650
2600
1952
1953
1954
1955
1956
1957
1958
Settlement Plate No. 12
Settlement Plate No. 3
Settlement Plate No. 8
Reservoir level

FIG. 4

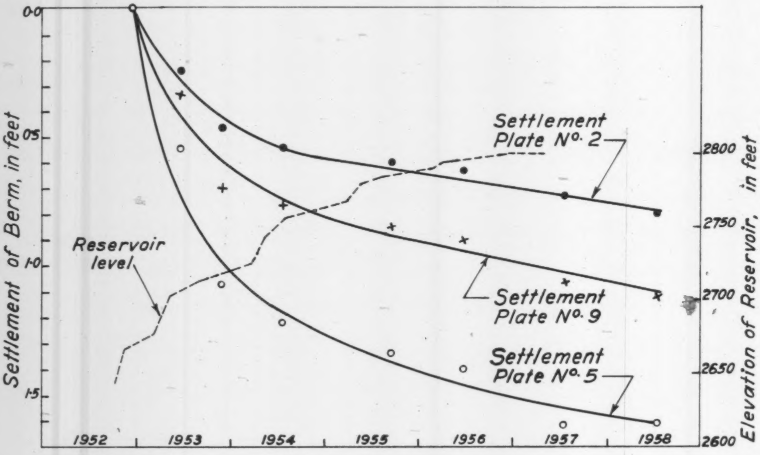
Settlement of Berm, in feet
0.0
0.5
1.0
1.5
Elevation of Reservoir, in feet
2800
2750
2700
2650
2600
1952
1953
1954
1955
1956
1957
1958
Settlement Plate No. 2
Settlement Plate No. 9
Settlement Plate No. 5
Reservoir level

FIG. 5

Observations have also been made to determine whether there has been any movement of the fill in a direction parallel to the axis of the dam, using the same monuments as for settlement and deflection. Table 7 indicates measurements of longitudinal movement for the crest and Table 8 corresponding movements for the berm. Pertinent comments are:

1. There is no apparent correlation between longitudinal movement and height of rockfill for either the crest or the berm.
2. Actual movements appear to be a maximum near the right abutment and a minimum near the centre. That is, these movements appear to be at a maximum where there is a minimum height of rockfill and a minimum where there is a maximum height of rockfill. This appears to be logical because the movement, while caused by a resultant of several forces, is undoubtedly influenced by the cross section of the valley at the site and possibly by the procedure followed in placement of fill. In

TABLE 3.—KENNEY DAM SETTLEMENT OBSERVATIONS, CREST IN FT

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov.20-23 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Settlement to July,1958,as percentage of height of fill
1	0	0.07	0.09	0.11	0.09	0.02	0.18	0.00	0
2	0	0.19	0.22	0.33	0.35	0.42	0.48	0.43	0.50
3	0	0.29	0.51	0.58	0.63	0.66	0.83	0.83	0.65
4	0	0.42	0.73	0.83	0.90	0.82	0.74	0.90	0.54
5	0	0.77	0.92	1.03	1.16	1.32	1.46	1.57	0.81
6	0	0.84	1.07	1.20	1.33	1.51	1.65	0.68	0.63
7	0	0.59	1.10	1.23	1.35	1.50	1.67	1.73	0.56
8	0	0.58	1.05	1.18	1.32	1.43	1.60	1.63	0.67
9	0	0.46	0.86	0.98	1.07	1.13	1.33	1.29	0.61
10	0	damaged	damaged	damaged	damaged	damaged	damaged	damaged	-
11	0	0.31	0.55	0.62	0.67	0.83	0.90	0.89	0.60
12	0	0.21	0.33	0.38	0.39	0.49	0.57	0.53	0.44
13	0	0.15	0.19	0.23	0.22	0.23	0.36	0.30	0.42
Average settlement									0.54

TABLE 4.—KENNEY DAM SETTLEMENT OBSERVATIONS, BERM AT EL. 2785, IN FT

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov.20-23 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Settlement to July,1958 as percentage of height of fill
1	0	0.12	0.20	0.25	0.27	0.31	0.36	0.26	0.50
2	0	0.24	0.47	0.55	0.60	0.63	0.73	0.80	0.87
3	0	0.32	0.66	0.76	0.83	0.89	1.07	1.06	0.81
4	0	0.43	0.88	1.00	1.09	1.14	1.31	1.31	0.84
5	0	0.55	1.07	1.22	1.33	1.40	1.61	1.61	0.70
6	0	0.64	1.12	1.26	1.37	1.53	1.66	1.61	0.59
7	0	0.47	1.01	1.13	1.24	1.31	1.44	1.49	0.72
8	0	0.40	0.84	0.96	1.05	1.08	1.24	1.22	0.70
9	0	0.33	0.70	0.77	0.85	0.90	1.07	1.12	0.82
10	0	0.30	0.53	0.61	0.67	0.72	0.79	1.00	0.89
11	0	0.18	0.30	0.36	0.39	0.41	0.53	0.96	1.08
Average settlement									0.77

the case of Kenney dam, with the exception of the upper portion, "---- the rockfill was built out from the left abutment in lifts approximately 40 ft. high, with each load sluiced down the slope from the top of the lift by skid mounted monitors delivering about two parts of water to one part of rock." It would appear logical any longitudinal movement in the fill should reveal itself to the maximum extent near the right abutment.

3. The movements appear to have attained a maximum by July, 1957, as would be anticipated due to the hydrostatic loading having reached a maximum that year.

TABLE 5.—KENNEY DAM DOWNSTREAM DEFLECTION, CREST IN FT

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov. 20 1953	July 15-17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Downstream deflection as a percentage of height of fill
1	0	0.01	0.06	0.02	0.03	0.04	0.10	0.19	0.38
2	0	-0.03	0.16	0.18	0.20	0.12	0.33	0.15	0.17
3	0	0.18	0.36	0.33	0.41	0.49	0.55	0.58	0.46
4	0	0.25	0.43	0.44	0.65	0.62	N.A.	0.71	-
5	0	0.39	0.58	0.72	0.85	0.83	1.08	0.98	0.56
6	0	0.36	0.81	0.79	0.91	0.90	1.16	-	-
7	0	0.49	0.85	0.83	0.98	0.93	1.23	1.04	0.34
8	0	0.39	0.69	0.73	0.90	0.97	1.11	0.76	0.31
9	0	0.30	0.62	0.62	0.76	0.81	0.95	0.60	0.28
10	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	-
11	0	0.24	0.41	0.34	0.43	0.57	0.56	0.66	0.45
12	0	0.10	0.31	0.19	0.36	0.37	0.32	0.30	0.25
13	0	0.01	0.20	0.11	0.12	0.16	0.15	0.26	0.36
Average deflection									0.36

Note:- (-) denotes upstream deflection

TABLE 6.—KENNEY DAM DOWNSTREAM DEFLECTION, BERM AT EL. 2785, IN FT

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov. 20 1953	July 15-17 1954	Sept. 1955	June 1956	July 19 1957	July 19 1958	Downstream deflection as a percentage of height of fill
1	0	-0.27	-0-49	-0.28	-0.34	0.28	-0.27	-0.30	-0.58
2	0	0.17	0.22	0.86	0.33	0.31	0.47	0.38	0.41
3	0	0.15	0.35	0.36	0.50	0.56	0.64	0.63	0.48
4	0	0.21	0.55	0.68	0.72	0.77	0.84	0.88	0.56
5	0	0.39	0.67	0.75	0.88	0.95	0.99	-	-
6	0	0.27	0.55	0.56	1.38	1.48	0.91	-	-
7	0	0.22	0.63	0.55	0.73	0.78	0.90	1.03	0.50
8	0	0.28	0.57	0.54	0.77	0.81	0.83	0.98	0.56
9	0	0.22	0.46	0.48	0.40	0.43	0.73	0.54	0.39
10	0	0.18	0.38	0.38	0.50	0.52	0.51	0.43	0.38
11	0	0.14	0.20	0.32	0.28	0.34	0.36	0.40	0.45
Average deflection									0.47

Note:- (-) denotes upstream deflection

The settlement, deflection and longitudinal movements given in Tables 3 to 8 are relatively erratic, those though possessing broad over-all consistency, as would be expected from consideration of the behavior of the fill materials.

Generally speaking, Kenney dam appears to have reacted to the imposition of full hydrostatic loading in the same manner as other well-constructed rockfill dams of modern execution and similar proportions. Its performance adds materially to confidence in the sloping impervious-membrane type of rockfill dam exemplified by the original Nantahala design.

TABLE 7.—KENNEY DAM LONGITUDINAL MOVEMENT AT CREST, IN FT

Settlement Plate No.	Dec.12-13 1952	Nov. 20 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Longitudinal movement as percentage of height of fill
1	0	0.01	0.04	0.09	0.12	-0.03	0.18	0.36
2	0	0.16	0.20	0.32	0.45	0.24	0.55	0.64
3	0	0.20	0.24	0.35	0.33	0.29	0.43	0.34
4	0	0.31	0.29	0.33	0.35	N.A.	0.44	0.26
5	0	0.23	0.04	0.37	0.31	1.22	0.20	0.10
6	0	0.16	0.16	0.30	0.15	1.11	0.30	0.11
7	0	0.06	0.09	0.36	0.51	0.98	0.39	0.13
8	0	-0.07	0.00	0.12	0.18	0.83	0.04	0.02
9	0	-0.09	-0.04	0.14	0.35	0.77	-0.05	0.02
10	0	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	-
11	0	-0.13	-0.04	0.21	0.13	0.69	0.05	0.03
12	0	-0.08	-0.03	0.16	0.11	0.67	0.13	0.11
13	0	-0.08	0.02	0.14	0.20	0.75	0.26	0.36

Note:- (-) All longitudinal movements are in the direction right abutment to left abutment except where indicated by (-).

TABLE 8.—KENNEY DAM LONGITUDINAL MOVEMENT OF BERM AT EL. 2785, IN FT

Settlement Plate No.	June 22 1953	Nov. 20 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Longitudinal movement as percentage of height of fill
1	0	0.63	-	0.60	0.68	0.44	0.61	1.18
2	0	0.59	-	0.58	0.63	0.39	0.68	0.74
3	0	0.49	-	0.52	0.50	0.37	0.60	0.46
4	0	0.45	-	0.50	0.54	1.28	0.46	0.29
5	0	0.39	-	0.37	0.31	1.14	-	-
6	0	0.26	-	0.24	0.19	1.06	-	-
7	0	0.19	-	0.15	0.11	0.92	0.03	0.01
8	0	0.12	-	0.08	0.12	0.86	0.03	0.02
9	0	0.08	-	0.12	0.15	0.78	0.24	0.18
10	0	0.03	-	0.06	0.04	0.79	0.18	0.16
11	0	0.00	-	0.06	0.10	0.70	0.10	0.11

Note: All longitudinal movements are in the direction right abutment to left abutment

WILLIAM G. HUBER,[6] **F. ASCE.**—The discussion by Mr. Cooke on Kenney Dam and on the movements within a rockfill, and the discussion by Mr. Lawton on detailed settlement of Kenney Dam present useful information that does not require further comment by the writer. The paper, its discussions, and the paper by D. J. Bleifuss and James P. Hawke[7] give full coverage of Kenney Dam.

A most valuable and comprehensive paper on geology, design, construction and performance of Cheakamus Dam has been presented by Karl Terzaghi, Hon. M. ASCE.[8] Cheakamus Dam was completed at the end of 1957. Its performance in terms of settlement and leakage during its first 2 yr of operation is excellent. As reported in Mr. Terzaghi's paper,

> "The settlement of the crest of the dam is a maximum above the old river channel where the height of the embankment is greatest (90 ft). At that point, the settlement of the crest increased during the first two years after completion of the dam at a decreasing rate from zero to 0.12 ft and the ultimate settlement will hardly exceed 0.25 ft . . . . The total loss of water due to leakage at full reservoir is about 0.7 cfs which is inconsequential. Therefore, it was justified to make the bulk of the impervious section of the dam out of relatively inexpensive, unprocessed Rubble Creek Wash. All of the water coming out of the contact layer beneath the rockfill section of the dam enters an inverted filter layer and the discharge of the springs coming out of the east slope of the river channel downstream from the dam is practically independent of the elevation of the water level in the reservoir. Therefore, there is no danger of a failure of the dam by piping."

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[6] Cons. Engr., Vancouver, B. C.

[7] "Rockfill Dams: Design and Construction Problems," by D. J. Bleifuss and James P. Hawke, Transactions, ASCE, Vol. 125, Part II, 1960.

[8] "Storage Dam Founded on Land Slide Debris," by Karl Terzaghi, Journal, Boston Soc. of Civ. Engrs., January, 1960.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3072

ROCKFILL DAMS: DESIGN AND CONSTRUCTION PROBLEMS

By D. J. Bleifuss[1] and James P. Hawke,[2] Fellows ASCE

With Discussion by Messrs. I. C. Steele; and D. J. Bleifuss and James P. Hawke

SYNOPSIS

This paper is confined to a discussion of rock fill dams having impervious zones of earthen materials. Terminology for various zones is suggested. The importance of thorough investigation of sites and materials is stressed, and reasons given from the standpoints of the designer and of the contractor. Design theories are lightly touched upon, since there is already considerable technical literature available on these. Construction problems are discussed; scheduling, river handling and choice of equipment are mentioned, and foundation treatment and placing of materials in the various zones are considered in some detail.

DEFINITION AND TERMINOLOGY

It is first necessary to define what is meant by the term "rock-fill dam". A rock-fill dam is one in which the thrust of impounded water is carried by an embankment of rock or gravel. This rock or gravel will not be watertight; therefore a watertight zone or member must be provided. This may be composed of compacted impervious compacted embankment, or it may consist of concrete slabs or steel. An impervious zone of compacted embankment requires transition zones between it and the rock-fill, so that the materials composing it cannot migrate into the rock-fill. The impervious zone must also be protected and weighted down on its upstream side, and transition zones are necessary on its upstream side also, so that its materials cannot migrate into the weighting zone. The writers intend to confine this paper to the discussion of rock-fill dams having an impervious zone of earthen materials.

The following terminology is suggested to the profession (See Figure 1):

1) Main rock-rill—The downstream rock-fill which supports the impervious zone and therefore carries the thrust of impounded water.

Note.—Published essentially as printed here, in October, 1954, as Proceedings Paper 514. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Cons. Engr., Atherton, Calif., Formerly Pres., Bleifuss, Hostetter and Assocs., Sacramento, Calif.

[2] Kaiser Engrs., Oakland, Calif., Formerly Chf. Engr., J. H. Pomeroy and Co., Inc., Los Angeles, Calif.

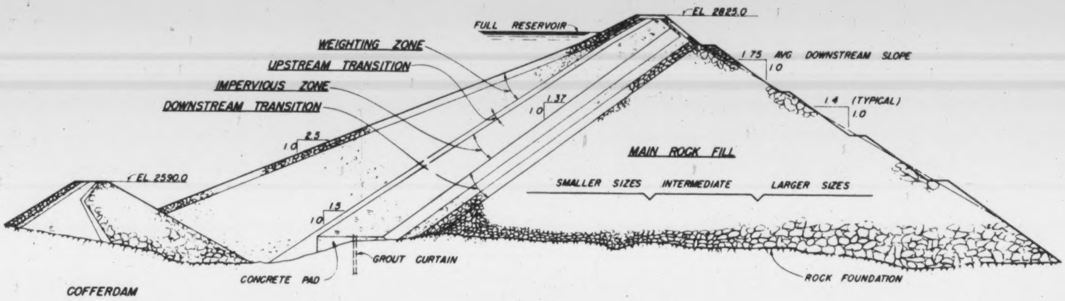
FULL RESERVOIR
EL 2825.0
WEIGHTING ZONE
UPSTREAM TRANSITION
IMPERVIOUS ZONE
DOWNSTREAM TRANSITION
1.75 AVG DOWNSTREAM SLOPE
1.0
1.4 (TYPICAL)
1.0
1.37
1.0
2.5
1.0
EL 2590.0
MAIN ROCK FILL
SMALLER SIZES INTERMEDIATE LARGER SIZES
1.5
1.0
GROUT CURTAIN
CONCRETE PAD
COFFERDAM
ROCK FOUNDATION

FIG. 1.—MAXIMUM CROSS SECTION OF KENNEY DAM

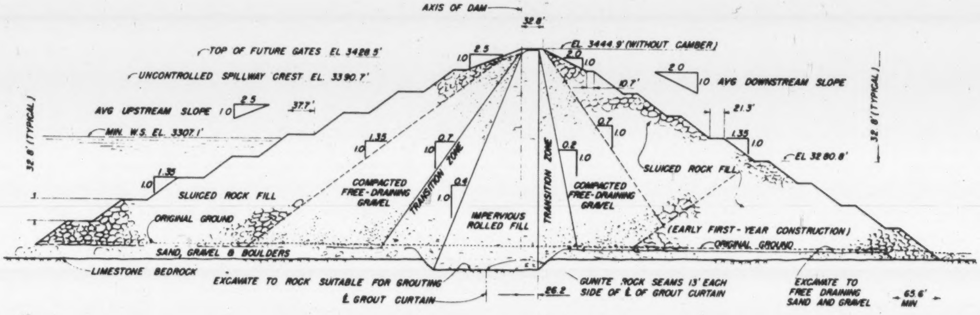
AXIS OF DAM
32.8'
TOP OF FUTURE GATES EL 3428.5'
EL 3444.9' (WITHOUT CAMBER)
UNCONTROLLED SPILLWAY CREST EL. 3390.7'
2.5
1.0
2.0
1.0
2.0
1.0 AVG DOWNSTREAM SLOPE
10.7'
AVG UPSTREAM SLOPE 1.0 2.5
37.7'
32.8' (TYPICAL)
MIN. W.S. EL. 3307.1'
21.3'
1.35
1.0
0.7
1.0
0.2
1.0
0.7
1.0
1.35
1.0
EL 3280.8'
SLUICED ROCK FILL
1.35
1.0
SLUICED ROCK FILL
COMPACTED FREE-DRAINING GRAVEL
TRANSITION ZONE
0.4
1.0
IMPERVIOUS ROLLED FILL
TRANSITION ZONE
COMPACTED FREE-DRAINING GRAVEL
ORIGINAL GROUND
(EARLY FIRST-YEAR CONSTRUCTION)
ORIGINAL GROUND
SAND, GRAVEL & BOULDERS
LIMESTONE BEDROCK
EXCAVATE TO ROCK SUITABLE FOR GROUTING
℄ GROUT CURTAIN
26.2
GUNITE ROCK SEAMS 13' EACH SIDE OF ℄ OF GROUT CURTAIN
EXCAVATE TO FREE DRAINING SAND AND GRAVEL
65.6' MIN

FIG. 2.—MAXIMUM CROSS SECTION OF KAJAKAI DAM

2) Downstream transition—The various zones between the main rockfill and the impervious zone which support and prevent migration of the materials of the latter. These transition zones are often referred to in engineering literature as "filters".
3) Impervious zone—That portion of the dam which provides watertightness.
4) Upstream transition—The various zones which lie between the impervious and the weighting zone, and prevent the latter from sinking into the former, and prevent the former from washing out into the latter during drawdown.
5) Weighting zone—The upstream portion of the dam, which weights or supports the impervious zone in place and protects it.

Importance of Site Investigation

The first step, of course, toward the solution of design and construction problems is comprehensive and thorough investigation of all phases of a proposed project. It is already well recognized that proper design is dependent on such investigation; it is not yet so generally recognized that sound and economical construction is equally so dependent. This should be emphasized. Investigation will be discussed briefly first, as it necessarily precedes both design and construction.

In order to provide a sound and economical design, the designer must know:

A. The characteristics of the dam site.
  1) Surface topography.
  2) Subsurface conditions.
  3) Hydrological data; how much runoff is available, and what the maximum flood is.

B. Characteristics of borrow pits or quarries.
  1) Volumes and locations of different kinds of material available, inasmuch as a rock-fill dam will ordinarily require rock, gravel, sand and impervious materials.
  2) The nature of each different kind of material, i.e., its structural and physical properties.

Either the site topography or the amount of runoff available will determine how large the dam can be; economic considerations will probably determine how large it should be. It is also conceivable that subsurface, i.e., foundation, conditions might bear a large part in determination of size. It might also be that amounts of material reasonably available would require consideration, as affecting economics, since while it may be said that is always plenty of material, it may lie too far away to be used. It is essential to know or estimate the peak and volume of the maximum flood, and under what conditions this flood is apt to occur, since good practice dictates provision of sufficient flood capacity so that the dam can never conceivably be overtopped. Parenthetically, while the writers generally believe and conform to this dictate, they also believe that a well designed and well constructed rock-fill dam will stand overtopping to a moderate degree without damage, as indicated years ago during the construction of Dix River Dam, and by recently reported experiences in Mexico.

The characteristics of borrow pits and quarries will bear a large part in the determination of the type of dam. No impervious material may be available, and consequently the dam may have to be faced with concrete slabs, as has been the case with some of the Pacific Gas and Electric Company dams in California.

If suitable earth materials which can be compacted to produce low permeabilities are available, an inclined impervious zone should be first considered. If, however, such materials are structurally weak, an impervious zone approaching a vertical position may have to be considered, for technical reasons appearing later herein.

In order to estimate costs, and to plan construction properly, the prospective builder must first have the design, and secondly a knowledge of the dam site characteristics and borrow pit or quarry characteristics. These will determine his construction schedule and affect his selection of construction equipment.

The writers wish to emphasize the importance of having investigations of earthen materials done by competent and experienced engineers. They have in mind one case where investigators on a foreign job reported that pervious materials were plentiful and impervious materials were very difficult to find, and the dam was designed accordingly. When the American engineers and construction crew arrived on the job, a thorough investigation program was carried out, and it was found that the exact opposite was the case; there was plenty of impervious material, and pervious material was scarce and expensive. It was necessary to redesign the dam, and this fortunately was entirely practicable, since the construction plant provided happened to be adaptable. Under other circumstances such an error could have been extremely serious. In another case the owner's geologist had gone to considerable trouble to locate a deposit of geological "clay" in the mistaken belief that such clay was necessary for the core of a rock-fill dam. They were not discouraged by the fact that this clay deposit was a number of miles from the dam site, that it was overlaid with many feet of rock which would have required blasting, and that the clay was indurated to the extent that it would have been difficult if not impossible to break it down for use in the embankment. Fortunately our investigations disclosed ample quantities of impervious borrow material immediately adjacent to the dam site.

Design Problems

The first consideration in a designer's mind is how much runoff is available, and its distribution throughout the typical year. Bearing in mind the use to which the prospective dam is to be put, he determines the desirable reservoir capacity which in turn establishes the height of the dam. He then estimates the characteristics of the possible maximum flood and within the limits of the dam site topography he studies the effect of flood storage capacity with respect to required spillway capacity. The spillway can often be provided thru a saddle adjacent to the abutment and the excavated material used in the dam. This should be considered in locating the spillway.

Bed rock is of course the best foundation. However, a reasonably clean and well-compacted gravel may be used as a foundation for all zones except the impervious zone, if it has a shear resistance approximating that of rock fill. Rock fill usually has a shear coefficient greater than 0.7, and large scale triaxial tests of gravels at the South Pacific Division Laboratory of the Corps of Engineers have shown that a well-graded clean gravel may have a shear coefficient approaching unity (1) (2).

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3. Numerals in parentheses, thus (1), refer to corresponding items in the Bibliography.

Assuming bedrock within a reasonable distance, the impervious zone should make contact with it. If this is impracticable, either some other means of cutting off seepage under the dam must be adopted, or the seepage path made so long that water can move along it only very slowly. This latter is sometimes accomplished by placing an impervious blanket on top of the overburden upstream from, and in extension of, the impervious zone of the dam. The use of such a blanket may necessitate a flatter upstream slope to the dam than would otherwise be necessary.

It is always necessary to grout a rock foundation. No matter how sound rock appears, it may have cavities in unexpected places. The writers recall one case of grouting massive anorthosite, very sound appearing, in which all holes but one in a certain area refused grout. That one was 125' deep, and 1100 sacks of cement were used before refusal.

Stage grouting is usually prescribed, starting with shallow holes at low pressure, and then following with deeper holes and greater pressures. The final stage should have diamond drill holes of depth equal to about one-half the height of the dam, and grout pressures at the surface equal to or exceeding those caused by the final developed head of water.

The main rock-fill of an inclined impervious zone dam should have its downstream face on a slope natural to the material; this will usually be about 1.35 to 1.4 horizontal to 1.0 vertical. This slope, of course, is the easiest to build. It has been argued that such a slope would be seriously disturbed by an earthquake. Recently, however, a severe earthquake caused no serious damage to such a slope on a large dam in Chile. The writers have been instrumental in having experiments made on a shaking table at the University of California, and these indicate no serious damage to a natural downstream slope caused by an earthquake with a horizontal acceleration of 1.0 g, which is of course much more severe than the ordinary design allowance.

The Institute of Transportation and Traffic, of the University of California, has recently undertaken some very interesting research which happens to be pertinent to the design of the outer slopes of rock fill dams (5). It has investigated a virtually unknown field, the shear strength of cohesionless materials under transient loads of short duration, such as would be caused by an earthquake. Test results will be published in the near future. In general, they show that, under such transient loads, granular materials have substantially higher shear strengths than under slowly applied loads. This probably accounts for the slight damages observed in the shaking table tests mentioned, as well as in the case of the Chilean earthquake. If, however, a downstream slope flatter than the natural is deemed necessary, although the writers can see no reason at all why it should be, it is suggested that a series of flat berms with natural slopes between them be used to average out to the flatter slope desired (see Figures 1 and 2). Compacted gravel fill may be substituted for all or part of the main rock-fill. Its natural slope will be somewhat flatter than the natural slope of a rock-fill.

If an inclined impervious zone is to be used, the upstream surface of the main rock-fill should also be on its natural slope. This permits construction of the main rock-fill in advance of construction of the transition and impervious zones, and eliminates the expensive work involved in dressing to a flatter slope. Specifications should provide that if there is a chance for selective dumping, the finer materials should be dumped along the upstream edge of the rock-fill.

If a vertical or nearly vertical impervious zone is to be used, the transition zones and the impervious zone must be carried up at the same time (i.e., only slightly behind) as the main rock-fill. High lift dumping on the main rock-fill is thus debarred and this is a disadvantage.

The main rock-fill is the principal structural element of the dam. The rock-fill must be free draining; this means that only a limited percentage of clay or quarry fines is permissible. The rock must be sound. Rock which breaks in chunky fragments is obviously better than rock which is slabby, such as schist. The fewer sharp points there are to be broken off, the less the settlement will be.

The writers were also instrumental in having experiments performed at the University of California on small models of rock-fill dams to determine the inherent stability of a rock-fill with an inclined impervious zone. A small 2-foot high model withstood a load equal to that which would have been imposed by a liquid of specific gravity fifteen times that of water. This is because the resultant of the water pressure is directed downward and increases sliding friction on the foundation. These experiments give point to the idea many engineers entertain, that the rock-fill dam with an inclined impervious zone has greater reserve stability than any other type of dam.

Whether the impervious zone is to be inclined or vertical, it is necessary to provide transition zones on both sides of it. That on the downstream side is the more important; it serves as the final surface upon which the impervious zone rests, and through it the water pressure load is transmitted to the main rock-fill. The transition zone must consist of materials so graded that on the one side they are too coarse to be forced or to migrate into the main rock-fill, and on the other side they are too fine to permit the impervious zone materials to mingle with or pass through them. Usually it will be found that the downstream transition zone will be subdivided into three zones, each with a different grading of material. Each sub-zone is designed to conform to the criterion that its 15% size is at least 4 times as large as the 15% size, and not more than 4 times as large as the 85% size, of the layer above it. In designing these sub-zones it must be remembered how they are going to be built. Each sub-zone should be at least truck width, otherwise their placement is apt to be a very expensive operation (see Figure 3). If the main rock-fill is gravel, the transition zone might be a single and relatively broad band formed by selective placing of varying material, as in the cases of Kajakai and Arghandab dams in Afghanistan. Specifications should provide that the downstream transition zone be carefully compacted.

The impervious zone must be built of material capable of being compacted until it is watertight. The watertightness is measured by its coefficient of permeability. This should not exceed 20 feet per year under unity gradient, and in general will be considerably less. If this material is well graded, and has a good shear strength, there is no doubt that the zone should be inclined. If it is not structurally strong, and has a very flat angle of internal friction (as, for instance, a fat clay), it may be necessary to use a vertical impervious zone, because if inclined, an excessively large weighting zone may be required.

The most satisfactory core material combines high structural strength, necessary impermeability, and good workability for economical construction. Such a material may be likened to a concrete mix designed in a modern laboratory in which the grading is such that there is a just sufficient quantity of a given grain size to fill the voids in the next larger grain size. Our ideal impervious material would be a well graded mixture containing about 15 to 20 percent silt and clay size particles with the remainder grading through fine to coarse sand sizes into gravel sized material. Providing there is an excess of finer material to completely encase them, and they do not interfere with proper compaction, there is no objection to gravel sized rock in the impervious embankment.

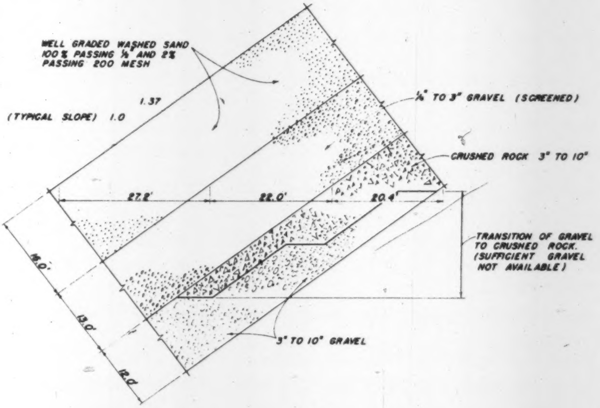
WELL GRADED WASHED SAND
100% PASSING ½" AND 2% PASSING 200 MESH
1.37
(TYPICAL SLOPE) 1.0
½" TO 3" GRAVEL (SCREENED)
CRUSHED ROCK 3" TO 10"
27.2'
22.0'
20.4'
TRANSITION OF GRAVEL TO CRUSHED ROCK. (SUFFICIENT GRAVEL NOT AVAILABLE)
18.0'
13.0'
12.0'
3" TO 10" GRAVEL

FIG. 3.—ENLARGED PARTIAL SECTION OF DOWNSTREAM TRANSITION ZONE OF KENNEY DAM

An embankment consisting entirely of clay is to be avoided because of its low structural strength and the difficulties of proper moisture control, both of which tend to result in a more expensive dam.

Compaction to certain standards compatible with the material should be specified. It is better to do this in terms of a percentage of an optimum test density according to a designated test, than, for example, to specify in detail the number of passes to be made with a sheepsfoot roller of a certain weight.

The specifications must cover with the greatest of care the matter of the contact between the impervious zone and bed rock, this being the most critical area of the dam. Rock must be trimmed so that there are no overhangs, no great sudden variations, and so that as the impervious zone moves under pressure it must occupy a smaller instead of a larger space.

The upstream transition zone must be composed of materials on one side fine enough so that they will not sink into the impervious, and on the other side coarse enough so that they cannot be washed into the weighting zone. It does not particularly need to be compacted.

The weighting zone should be composed of heavy material with a fair angle of internal friction, say 35 degrees. Most dams are subject to drawdown, and on this account the material should be free-draining. Otherwise, saturated material above the free water surface would increase the liability of slides occurring.

The downstream surfaces of dams must be protected against erosion; a rock-fill needs no special treatment, but a gravel fill will need a surface layer of rock. The upstream surface must be protected by riprap against wave action, unless the weighting zone material is composed of pieces individually heavy enough to provide suitable protection, as determined by recently published design criteria (3) (4).

Since a rock-fill dam, when the water load comes on it, will settle in two directions, vertically and downstream, it follows that in general plan the dam must be convex upstream, and in general profile must be crowned, that is, made higher in the middle of the dam than at the abutments. These convexities should be at least three times the anticipated amount of motion. The settling then results in a crowding of the fill into a smaller volume.

There is another consideration which makes it advisable to put a vertical crown, or camber, on the crest of the dam, and that is appearance. A truly straight line will appear curved to the eye. A perfectly horizontal crest will appear to sag in the middle. This is a well-known fact; the ancient Greeks knew it, and built the Parthenon accordingly some 440 years B.C. There are no long straight lines in the Parthenon. It is cambered both longitudinally and transversely.

There are two classical methods available for the stability analysis of rock-fill dams. The best known is that of Fellenius, involving the assumption of curved failure surfaces, with thin slices taken along these surfaces for purposes of computation. Neglecting the shearing forces on the vertical faces of the slices, the forces tending to cause motion are compared with those tending to resist motion. In a variant of this method, called the method of planes, the failure surfaces are assumed as planes instead of curves.

The second classical method is the "sliding block" method, in which the main rock-fill and the weighting zone are considered as retaining walls confining and supporting the impervious zone. The impervious zone, the downstream transition, and the main rock-fill are also considered as resisting the hydraulic thrust of stored water.

The routine computations involved in either of the above methods are relatively simple, and are so thoroughly described in technical literature they will not be further discussed here. However, their practical

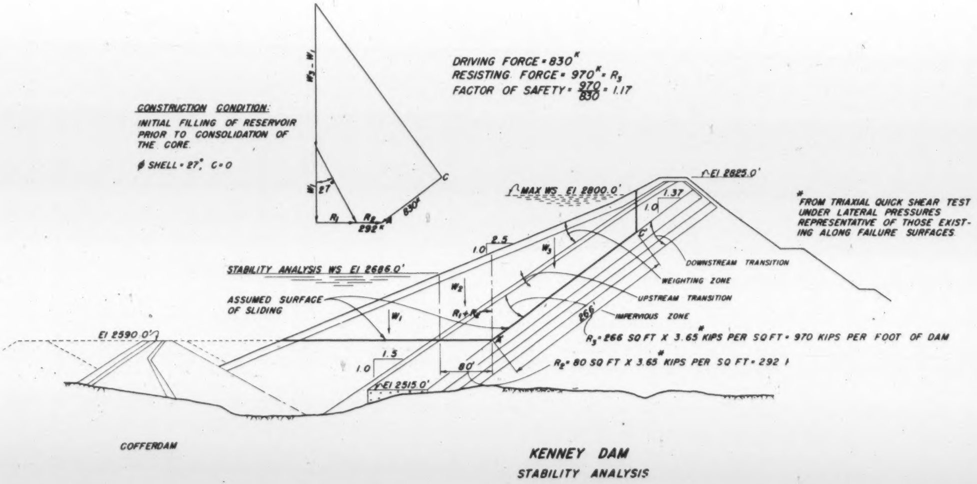
CONSTRUCTION CONDITION:
INITIAL FILLING OF RESERVOIR PRIOR TO CONSOLIDATION OF THE CORE.
ϕ SHELL = 27°, C = 0
$W_3 - W_1$
W_1
27
R_1
R_2
292^K
830^K
C
DRIVING FORCE = 830^K
RESISTING FORCE = 970^K = R_3
FACTOR OF SAFETY = 970/830 = 1.17
El 2825.0'
MAX WS El 2800.0'
1.37
1.0
2.5
1.0
W_3
C
* FROM TRIAXIAL QUICK SHEAR TEST UNDER LATERAL PRESSURES REPRESENTATIVE OF THOSE EXISTING ALONG FAILURE SURFACES.
DOWNSTREAM TRANSITION
WEIGHTING ZONE
UPSTREAM TRANSITION
IMPERVIOUS ZONE
266'
STABILITY ANALYSIS WS El 2686.0'
W_2
ASSUMED SURFACE OF SLIDING
W_1
$R_1 + R_2$
A
R_3 = 266 SQ FT X 3.65* KIPS PER SQ FT = 970 KIPS PER FOOT OF DAM
R_2 = 80 SQ FT X 3.65* KIPS PER SQ FT = 292 K
El 2590.0'
1.5
1.0
80'
El 2515.0'
COFFERDAM
KENNEY DAM
STABILITY ANALYSIS

FIG. 4.—STABILITY ANALYSIS OF KENNEY DAM

application to a composite dam section requires considerable judgment and experience.

In the case of Kenney Dam, Dr. Karl Terzaghi recommended an analysis using plane failure surfaces (see Figure 4). The factor of safety is computed as the ratio of resisting forces along the assumed failure surface, Line A' C', to the driving forces along Line AC in the force diagram. It will be noted that the driving forces are a function of the slope of Line A' C'. This method should be used only by engineers of considerable experience.

In studying Kajakai and Arghandab dams, which have nearly vertical impervious zones, the sliding block method of analysis was used, as well as the Fellenius method. The sliding block method necessitates a careful consideration of shearing strains, and it is likewise not for the tyro.

It should be borne in mind that stability analyses must be made for various conditions of drawdown. For dams with inclined impervious zones, the most critical condition usually occurs when the water surface is something less than halfway up the dam.

As a final comment, the designer's problem is to select a site, accepting its limitations, and design a satisfactory dam, using the materials at hand if practicable, with as little waste as possible.

Construction Problems

General

The writers would like to quote the approximate words of Col. J. P. Growdon, Mem. A.S.C.E.: "Materials exist in nature in chaotic form. Construction is the art of assembling them in an orderly fashion, in accordance with a preconceived plan." In practicing this art, many problems arise, and the authors intend to discuss a few which occur in rockfill dam construction.

Specifications can and must be provided which will cover the general treatment of construction. However, on every job, conditions arise which require special treatment. This points the necessity of having engineering personnel in the field with experience and good judgment. The construction of earth and rock fill dams cannot be prescribed under as definite specifications as that of masonry dams; proportionately greater judgment is necessary on earth and rock fill dams. It is not to be expected that field engineers can always be obtained who are competent to handle any and all special cases; it is absolutely essential that such personnel be able to recognize a case requiring special treatment. They can always call for help if in any doubt as to how to handle it. The writers can cite cases in which a serious condition was not immediately recognized, but was picked up during a visit of a superior, and such a state of affairs always leaves one wondering what else happened which was serious and not reported. This is a most uncomfortable feeling. It is certainly futile to maintain rigid inspection during day shifts and have little or no inspection at night.

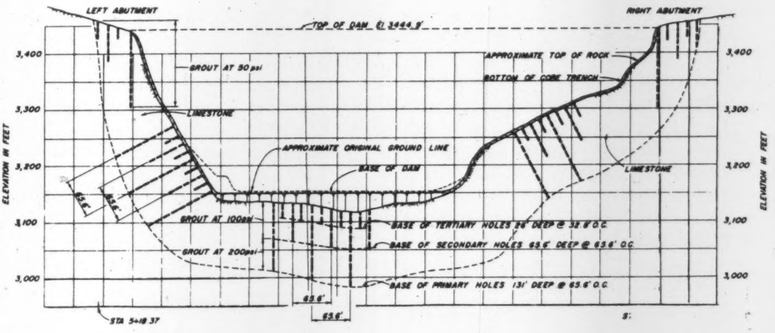
LEFT ABUTMENT
RIGHT ABUTMENT
TOP OF DAM El. 3444.9'
APPROXIMATE TOP OF ROCK
BOTTOM OF CORE TRENCH
GROUT AT 50 psi
LIMESTONE
APPROXIMATE ORIGINAL GROUND LINE
BASE OF DAM
LIMESTONE
65.6'
65.6'
GROUT AT 100 psi
GROUT AT 200 psi
BASE OF TERTIARY HOLES 26' DEEP @ 32.8' O.C.
BASE OF SECONDARY HOLES 65.6' DEEP @ 65.6' O.C.
BASE OF PRIMARY HOLES 131' DEEP @ 65.6' O.C.
65.6'
65.6'
STA 5+18.37
3,400
3,300
3,200
3,100
3,000
ELEVATION IN FEET

FIG. 5.—FOUNDATION GROUTING OF KAJAKAI DAM—PROFILE ALONG CENTERLINE OF CORE TRENCH

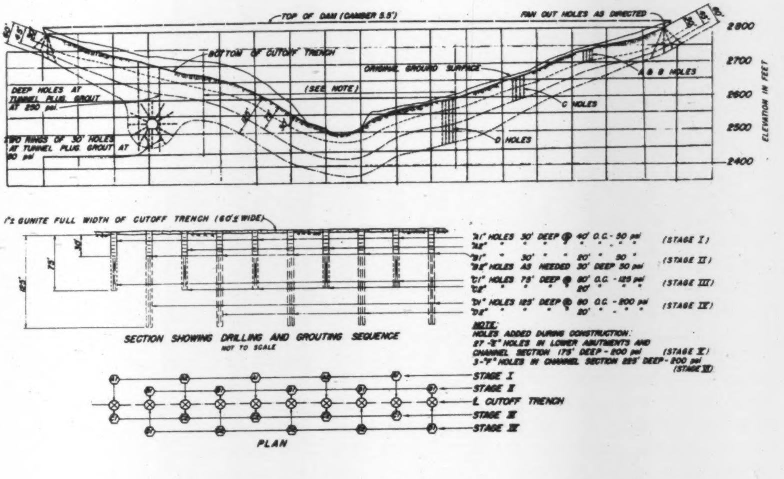
TOP OF DAM (CAMBER 5.5')
FAN OUT HOLES AS DIRECTED
BOTTOM OF CUTOFF TRENCH
ORIGINAL GROUND SURFACE
(SEE NOTE)
A & B HOLES
C HOLES
D HOLES
DEEP HOLES AT TUNNEL PLUG GROUT AT 250 psi
TWO RINGS OF 30' HOLES AT TUNNEL PLUG. GROUT AT 0 psi
2800
2700
2600
2500
2400
ELEVATION IN FEET
1'± GUNITE FULL WIDTH OF CUTOFF TRENCH (60'± WIDE)
30'
75'
125'
"A1" HOLES 30' DEEP @ 40' O.C. - 50 psi (STAGE I)
"A2" " " " " " " " "
"B1" " 30' " " 20' " 50 " (STAGE II)
"B2" HOLES AS NEEDED 30' DEEP 50 psi
"C1" HOLES 75' DEEP @ 80' O.C. - 125 psi (STAGE III)
"C2" " " " " 20' " " "
"D1" HOLES 125' DEEP @ 80 O.C. - 200 psi (STAGE IV)
"D2" " " " " 20' " " "
NOTE:
HOLES ADDED DURING CONSTRUCTION:
27-"E" HOLES IN LOWER ABUTMENTS AND CHANNEL SECTION 175' DEEP - 200 psi (STAGE V)
3-"F" HOLES IN CHANNEL SECTION 225' DEEP - 200 psi (STAGE VI)
SECTION SHOWING DRILLING AND GROUTING SEQUENCE
NOT TO SCALE
STAGE I
STAGE II
℄ CUTOFF TRENCH
STAGE III
STAGE IV
PLAN

FIG. 6.—PROFILE ON CENTERLINE OF GROUT CURTAIN OF KENNEY DAM

Schedule

The first problem facing the prospective builder is that of preparation of a schedule. He must know the approximate quantities, the characteristics of the dam site, and how the river is to be handled during construction. To avoid very expensive diversion facilities, it may be necessary that the entire dam be built in the period between one flood season and the next; in fact, it may be found that this is the only way the dam can be built. Ordinarily, there is a "natural" period of construction for every dam, to do it faster means useless expense in the way of excess equipment and inefficiency, and to do it slower means increased cost because the overhead goes on longer. Of course, an urgent need for water or power could necessitate the faster schedule regardless of expense. But usually, the river regime or the "natural" construction period will give the builder the skeleton of his schedule, and the detailed items merely go to clothe this skeleton.

There are in common use three methods of handling a river during construction:

a) Diverting it through a tunnel.
b) Building part of a dam while the river runs through the area to be occupied by the other part.
c) Fluming the water through.

Only (a) and (b) are applicable in the construction of a rock-fill dam, and (b) can never be completely applicable, since there usually must be some way of passing water while the last part of the dam is being built. The choice is dictated by the topography of the site. At Kajakai Dam the gorge was deep and narrow (see Figure 5), and there was no choice except to divert through tunnels. Only four years of stream flow record were available, and from these it was estimated that the spring flood peak would hardly be greater than 50,000 cfs. Two 26' horseshoe tunnels were driven, which would carry 50,000 cfs with a certain head at their inteakes. Construction of the dam would take two seasons, so it was necessary to get the dam high enough the first season to provide this head. This left the possibility that a greater flood might overtop the partially completed dam. The main rock-fill was therefore made higher than upstream portions of the dam, and selected large rock used on the downstream surface. It was thought that some slight overtopping would not wash out any of the rock-fill, and that the fill would cause a pool over the rest of the dam which would not therefore be damaged. Fortunately, the flood that year peaked at about 30,000 cfs, and the theory was never put to the test. A calculated risk had to be taken that an unusually large flood might wash out everything.

At Arghandab Dam, two seasons were likewise required for construction. But the dam was over 2000 feet long, so during the first season about one-half the dam was built while the river ran through its ordinary channel near the left abutment. After the spring flood, and during the second season, the river was diverted through a small tunnel while the rest of the dam was built. The tunnel had to carry only low water flow, and was required anyway for irrigation and power release upon completion of the project. There was hardly any risk at all involved, whereas there was an appreciable risk at Kajakai.

Kenney Dam was built during one season. This was essential because the reservoir created is enormous, and will require five or six years to fill. The gorge was narrow, (see Figure 6) and work was begun after the spring flood peak, so that only relatively low flow was required to be diverted, and it was handled through a tunnel with no trouble at all.

Having a schedule, and quantities, the builder has one item of information necessary to his selection of construction equipment, i.e., he knows the rates of speed at which various items of work must be done. He must next know where his materials lie, i.e., how far they have to be hauled, and up or down what grades. He must further know what the materials are; for instance, whether rock is apt to break large or small, since this would be a consideration in choosing equipment sizes. He must know how the materials lie; they may be in large deposits or in scattered small deposits; this might mean a few large pieces of equipment, or a greater number of small pieces. Also, in borrow pits of gravel, sand, or impervious material, characteristics may vary with depth, and this affects choice of equipment. For instance, a uniform deposit of large extent might be dug with elevating graders, whereas a small deposit might require power shovels. Or a non-uniform deposit might require averaging excavation over certain depths, and consequently power shovels would be required.

Again, a non-uniform deposit might require zoning, i.e., certain areas might contain material satisfactory for impervious zones, other areas might contain coarser materials satisfactory for other zones in the dam. The locations and sizes of these areas bear on the choice of equipment, and also on the order in which they are to be used.

The writers wish to call to the attention of builders that very often it would be advisable to get the advice of consulting engineers before selecting construction equipment. They have in mind an instance of a contractor having purchased a rather expensive well point system to protect excavation in a very well-graded mixture of gravel and sand which was almost watertight. It was found to be unnecessary, and a good soils engineer could have determined this from analyses of the material.

Foundation Treatment

Mr. I. F. Thompson, Affiliate A.S.C.E., in a paper "Foundation Treatment for Earth Dams on Rock" (7) has discussed the importance of proper preparation of the foundation on that area with which the impervious zone comes in contact. This importance cannot be overemphasized. In a rock-fill dam with an inclined impervious zone this contact area would be relatively narrower than in the earth-fill dams Mr. Thompson has discussed.

While some latitude may be allowed in cleaning foundations under the other areas of the dam, the area under the impervious zone must be cleared to material already actually in place which is impervious or can be made impervious by grouting. In the writers' experience such material has always been rock. It can be said here that no rock is to be regarded as impervious; there are always joints or bedding planes, or both, and all rock foundations must be considered as requiring grouting.

After thorough cleaning, it will usually be found that it is wise to fill some small local irregularities with concrete, which should be left with a rough (untrowelled) surface. In the Kajakai Dam in Afghanistan, on limestone, an old channel was uncovered full of pot holes. It was easier to concrete these pot holes than to trim off the sound rock projections between them, and it would have been a slow and expensive operation to hand fill the numerous small depressions with earth, particularly since many of them had overhanging edges.

The impervious zone of the dam must in effect be extended into the foundation so that water has to travel a long and circuitous path through small passages to get around it. There are joints which must be grouted, and bedding planes in case the rock is sedimentary. There may also be faults which must be rendered tight.

It is interesting to note that the associates of the writers have independently developed, and have used for some years, a treatment procedure similar to that recommended by Mr. Thompson. Typical treatment at Kajakai Dam is shown in Figure 5, and for Kenney Dam in Figure 6.

Kajakai Dam foundation is limestone. At Kenney Dam, on the Nechako River in British Columbia, the foundation consists of a series of layers of vesicular basalt and lavae, laid down at different times. In fact, during excavation, pieces of sound, although charred, wood were uncovered under lava flows. At both dams, the initial shallow series of holes were found very difficult to grout, because there were so many surface openings at which grout came out that pressure could not be built up at all. The late Mr. James B. Hays, Mem. A.S.C.E., was in charge of this work on both jobs, and finally decided to cover surfaces of areas to be grouted with a coat of gunite, averaging from 1/2" to 1" thick, except that small depressions were filled or rounded out so that hand (air) tampers could be used to fill them with impervious material (6). Pressure could then be built up and grouting done. Care was used to keep the grout pressures low enough to avoid heaving the foundation. This is important. Heaving of foundations under such conditions has been known to occur, and has necessitated some very expensive repairs.

With such a gunite covering of surface cracks, grout injected into any hole is forced to travel some distance before it can escape. This means more pressure required, but a greater range is grouted from each hole, both at surface and at depth. Surface cracking of the gunite did not amount to much, although at Kenney some of it was exposed for a year, through a temperature range from 20 degrees below zero to 80 degrees above, Fahrenheit.

At Kajakai Dam, an almost vertical fault was discovered, running almost at right angles to the axis of the dam, slightly over to one side of the buried channel previously mentioned. It was obviously inactive, about eight feet wide, and filled with very tightly packed gouge of clay, broken limestone, and small solution deposits. The proposed static head at the rock surface was nearly three hundred feet. Mr. John S. Cotton, M. A.S.C.E., and the senior writer agreed on a shaft about 15 feet square, centered on the fault, sunk to a depth of 100 feet, and filled with concrete. Grout pipes were left in this concrete, but very little grout could be forced into them. Two or three other smaller faults were similarly treated.

Before grouting, every effort should be made to clean out clay-filled seams by washing. If it is impracticable to do this, and there appears any possibility that the material left in the seams might some day migrate, it is a good idea to provide places near the toe of the dam where water can escape from such seams. These escapes must contain inverted filters, so that, while water can escape, it can carry no fine material with it. If the fine material cannot get out, there will be no piping, and leakage will not tend to increase with time. If it can get out, a strong flow of water may develop, and the repair operation can be pretty difficult and expensive.

A very good contact must be obtained between the impervious zone and rock foundation. The impervious material must be hand tamped all over the contact area. There must be no continuous seepage path along the contact. Probably no better job of hand tamping has been done than at Kajakai and Arghandab dams in Afghanistan, where the work was done by tribesmen under their own foremen.

The impervious contact area must first be cleaned thoroughly in order to find out what must be done to it; after all filling, grouting, and other work is done, it must receive a very careful final cleaning just before impervious material is placed. Literally everything except solid sound rock must be removed. Air blasts are preferable to water jets, since the latter are bound to leave small pockets and pools of water which must be dried up before any material is placed.

Our problem which must always be settled in the field is how far to go in trimming and shaping the rock foundation under the impervious contact. It has been mentioned that it is probably advisable to fill small inequalities with concrete. Whether at the bottom of the dam site or along the abutments, the same general principles hold. It is neither necessary nor desirable that the contact area be strictly planar, but there must be no abrupt changes of section, and there must be no overhang. It is obvious that material settling around an overhang must leave a cavity beneath it. Abutments for the inclined impervious zone dam must be so trimmed and sloped that when load comes on the dam, and the impervious zone moves downward and downstream in consequence, as it must and will, the impervious material always tends to move into a smaller, and not a larger space. This must be watched very closely.

While in the impervious contact areas the main problem is watertightness, under the load-carrying portion of the dam we are interested in two other things instead, settlement and friction. The main rock-fill must be built so that when the water load comes on it the settlement is as little as possible, in order to minimize the adjustments which must take place in the impervious zone. The main rockfill must therefore be placed on a firm foundation of rock or good gravel. Silt or clay generally is not permissible, and must be removed. Ordinarily, and particularly in the case of dams with inclined impervious zones, the friction between the rock-fill and the foundation will be found to be several times that required to prevent the dam from sliding. The field Engineer must use his judgment as to what shall be removed.

Main Rock-Fill

The problems in connection with the main rock or gravel fill begin at the quarry or borrow pit. The quarry must be drilled and shot to produce rock of sizes which can be handled by the equipment, with only a small quantity of extreme fines. Some secondary "pop" shooting of large rocks must be expected; if there is not a little of this, it will be found that the rock is being shot too fine, and too much explosive is being used. At Kajakai, limestone rock was obtained from spillway excavation. Wagon drills were used. At the start, 0.75 pounds of 40% explosive was used per cubic yard of solid rock; it was immediately obvious that this was a little too much, and thereafter not more than 0.625 pound was used. At Kenney Dam, there was no real good quarry site within any reasonable distance. Consequently, a quarry site was chosen about 1/4 mile to the left of the dam, and upstream. The rock was basalt, with some lava, and was very spotty, masses of sound rock with badly weathered areas interspersed. Quarry depth was limited by the depth of the basalt, which lay on older and unusable materials. The contractor became committed to the use of "coyote" holes, and this was a constant source of trouble, because good rock, unsound rock and clay became mixed up; also the amount of explosive used in one shot was no index to the amount to be used in the next, so that practically all blasts were a little undershot or a little overshot. It is believed that drilling with quarrymasters would have allowed more selective shooting, and would have simplified the sorting of material as it was loaded into trucks. In general, about 0.9 pound of explosive was used per solid cubic yard.

The main rock (or gravel) fill should preferably be absolutely free-draining, and the material should therefore be clean. Practically, however, it may be economically impossible to get absolutely free-draining material, in which case the writers believe it should have a permeability at least twenty times that of the material in the impervious zone. In the case of Nantahala Dam, the rock (arkose) was clean, and contained only a limited amount of fines. In the case of Kajakai, the rock (limestone) was also clean, and if not shot too hard, did not contain too many fines, and gravel pits were carefully zoned and watched, only the coarsest being placed in the main rock-fill. In the case of Kenney Dam, clay and earth were so intermingled that it became necessary to decide how much dirt was permissible. Dr. Karl Terzaghi was consultant, and he agreed that no single load of rock containing more than 15% of dirt could be used in the main rock-fill. Since many loads were clean, the average would be much less than 15%. Inspectors were placed at the quarry face, watched the loading, estimated the percentage of dirt, and accepted or rejected the loads. From 25% to 30% of all loads were rejected; these were dumped over a 50-foot drop, and much good rock rolled to the bottom where it was reclaimed by a shovel working along the toe.

Notwithstanding this high percentage of waste, a little more than 4,000,000 cubic yards were placed in Kenney Dam between May 10 and November 27 of 1952, and the dam was completed in one season.

It is the writers' opinion that a certain amount of fines in a rock-fill act as a lubricant when water is applied, and permit better adjustment of individual rocks. Other engineers have expressed violent disagreement with this idea.

The writers believe it desirable to place rock-fill in as high lifts as possible, holding that the energy expended as rock moves down goes largely into compaction, and the greater the lift height the greater the energy. Other considerations may preclude making only one lift the full height of the dam. It may result in undesirable hauling conditions; it may be necessary to place the impervious zone before the main rock-fill is completely built; the top is usually too narrow to provide sufficient working room; some designers and governmental authorities may require slopes flatter than the natural. Again the writers wish to emphasize that this last requirement is not necessary.

The proper application of water during placing of rock-fill is all-important. There must be at least two cubic yards of water applied for each loose cubic yard of fill placed, and four cubic yards of water are better. There is no necessity for more than four. Water should be applied by hydraulic monitors in streams of 2-1/2 or 3 inches in diameter, and the nozzle pressure should be at least 75 p.s.i. The streams, or better, jets, should hit the rock as it is being dumped and immediately thereafter, and should be directed toward the face of the rock-fill slope, never parallel to or down it. On one dam with which the writers were connected, when starting the rock-fill the water was applied as shown in Figure 15. This is exactly the way not to do it. Water was directed down the slope in sprinkling fashion, and might as well have been applied with a garden hose. This method was very speedily revised. At Kenney Dam, a very satisfactory scheme was developed. Each monitor was mounted at one end of a pair of steel beams about 60 feet long. On the beams near the other end was mounted a diesel-driven compressor, furnishing air to the monitor controls. This assembly was shoved over the edge of the rock-fill so that the monitor end cantilevered some 25 feet out into the air, the compressor acting as a counterweight, and the jet could be played back against the face of the fill. The point at which the jet strikes must be varied a good deal, and for this reason mechanical controls are important. It is hard to control a monitor by hand, and it will not be moved enough unless power-controlled.

When water is properly applied, the effect on a mound of freshly dumped rock is amazing. It seems to dissolve and flatten out, and there is not much rolling of rock down the dump face. Rather there are frequent slides. Segregation of sizes is minimized. Settlement takes place during construction, and not more than 1.0%, or at the most 2.0%, may be expected thereafter.

The pumps for the water supply should be carefully specified by the engineers. Spares should be provided. The piping layout should be carefully designed for minimum head losses and for easy adjustment as the fill progresses. In this connection some weird things can happen. On one job, engineers under the writers' direction selected and ordered diesel-driven pumps, electric power in sufficient quantity not being easily available. The pumps were rated at 1750 RPM, and the diesels were furnished accordingly. On visiting the job, it was observed that the water supply was inadequate. Investigation revealed that a mechanic had concluded that 1750 RPM was too fast for a diesel, he thought it would wear out too quickly, and without saying anything to anyone, he had adjusted the governors to give 1250 RPM. A pump rated for a certain performance at 1750 RPM gives a pretty sickly performancy at 1250 RPM. This was pointed out, the governors were corrected, and the water supply was immediately satisfactory.

Transition Zone

The foundation under the downstream transition is generally prepared in the same manner as that under the impervious zone, except that there is no necessity for grouting. The transition will usually have three zones; one might be composed say of 10 to 3 inch crushed rock or screened gravel, the next of 3 to 3/4 inch crushed rock or screened gravel, and the third, against which the impervious is placed, of 3/4" to fine clean sand. These materials may not contain more than two percent of silt or clay. The clay or silt content can be easily determined by taking a handful of the finest zone material, well moistened, and clamping it in the fist. If on release the material falls apart, it is clean enough; if it remains in a lump, it is not. Of course, laboratory determinations should be made to check, or to teach inspectors how to make, such rough observations. At Kenney Dam, glacially deposited sands and gravels were used, and it was found that the screened sand contained an excessive amount of silt. A flat-bottomed steel trough was built on a 28 degree slope; at the lower end of this was placed a steel bin with a sand gate in its bottom, and provision for water overflow at its top. Sand was gradually pushed in at the upper end of the trough by bulldozer, numerous small jets of water played uphill against it, and the sand and water flowed down into the bin. Sand with about 1% of silt settled to the bottom, and the overflow water carried off the rest of the silt. Trucks ran under the bin for loading. It was quite a cheap and very effective operation.

The transition is usually deposited only slightly ahead of the impervious, and water cannot be used because some of it would get into the impervious and affect compaction. Only mechanical compaction is possible. Usually bull-dozers are kept moving back and forth along the transition. The various zones must each be of sufficient width to accommodate the trucks in use (probably 12' minimum); it simply costs more money to make them narrower.

Impervious Zone

The methods of placing and compacting earth-fill are so well-known that the writers will not discuss the placing of the impervious zone.

Moisture control is often best accomplished in the borrow pits. Kajakai Dam is in desert country, and it was necessary to terrace and dike the borrow pits and irrigate them well in advance. At Kenney Dam, the borrow pits were in glacial material, and water had to be applied. A scheme developed on the job worked rather well. A set of ripper teeth was mounted on a tractor in place of a bull-dozer blade, and behind each tooth a pipe injected water at a depth of about 18 inches. The material was subsequently mixed by repeated harrowing.

As Kenney Dam was nearing completion, everyone began to worry about frost. It was nip and tuck whether the dam could be completed before winter set in, and it would have been very expensive to start up again in spring for only a very small operation. So tests were made of admixtures to prevent freezing. It was found that the addition of common salt, in the proportion of 1.0% by weight, of soil, would offer satisfactory protection. The salt was spread dry over the borrow area and harrowed in before adding water. When salt was so added, less water was required. The fall turned out to be mild, and not enough frost was experienced to be able to evaluate fully this salt treatment.

Upstream Transition Zone

The foundation preparation for the upstream transition zone is generally similar to that of the area under the main rock-fill. The zone material should be placed in the same manner as that in the downstream transition zone. It should be clean and free-draining.

Weighting Zone

The weighting zone, as has been stated, is for the primary purpose of preventing the impervious zone from sliding down and out upstream during initial filling of the reservoir or subsequent drawdown. The term "sudden drawdown" is often used. The writers wish to protest this unnecessary term, since a properly constructed impervious zone drains so slowly that all drawdowns are sudden, whether they take one day or one year. The weighting zone material should be as heavy as possible, and it should be free-draining, so that pore pressure does not reduce sliding resistance. Cleaning of the foundation area is not particularly important, but of course weak materials which might permit upstream sliding should be removed. The new material does not need to be compacted when placed.

The surface of the weighting zone should be covered with material which will not wash under rains and which will be resistant to wave action.

CONCLUSIONS

Engineering literature on rock-fill dams is still rather scanty. The writers have attempted to discuss the problems of practice which beset the designer, as well as some of the field problems which have a way of bursting suddenly on the unwary field engineer.

Engineering problems in the field are quite often the sort not solvable by reference to books, and hence their solution requires sound judgment based on experience. When dealing with a rock-fill dam, its design likewise is not a matter of reference to books, and exact mathematical formulae cannot be applied. Therefore a rock-fill dam requires the exercise of judgment more than many other structures.

Points to be emphasized in design and construction are:

a. The necessity for competent and thorough investigation of site and materials.

b. The necessity for having engineering personnel in the field during construction able to recognize a situation requiring special attention.

c. The necessity for close attention to foundation preparation.

d. The necessity for proper sluicing in placing rock fill, so that settlement after construction will be minimized.

e. The necessity for careful placing of materials in transition and impervious zones.

Since only very general requirements can be laid down in advance, only an engineer experienced in both design and construction can say when investigation of sites and materials has been adequate in any specific case. The same may be said of foundation preparation. An engineer with substantial field, laboratory, and design experience is required to determine whether the various materials are being properly placed and compacted.

APPENDIX I.—REFERENCES

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DISCUSSION

I. C. STEELE,[4] F. ASCE.--Mr. Bleifuss and Mr. Hawke are to be complimented on the excellence of their paper. There should be many more papers of a similar nature upon the subject of the Design and Construction of Rockfill Dams of all types.

The authors have confined their comments to the composite structure consisting of an impervious zone of earthen materials with rockfill or gravel supporting sections. This composite structure is of two types, the inclined core and the vertical central core.

The writer's experience on rockfills, until recent years, has been chiefly with what he chooses to call the true rockfill type of dam with an upstream face membrane of concrete, timber or gunite. Pacific Gas and Electric Company, with which he was associated for many years, has 14 such dams on its system.

The two composite types and the true rockfill type, when properly designed and constructed, make splendid and durable dams. Selection of the type for a specific site should be determined by site conditions such as character of foundations, available materials, economics and other factors. In some locations, such as in the higher reaches of the Sierra Nevada Mountains in California, impervious material either is non-existent in quantity or too costly to obtain. At such sites the impervious face type of rockfill has proven considerably more economic than concrete dams.

The authors have ably described design and construction requirements for inclined and vertical core dams.

Settlement will occur in both types during construction and when reservoir pressures are applied. There will be significant lateral settlement, from the abutments toward the center of a rockfill, as well as the better known vertical and horizontal movements. Lateral settlement and differential settlements due to abrupt topography changes require careful consideration in order to avoid or minimize cracks. For these reasons, the rockfill should be constructed to give minimum settlements throughout the dam rather than to reduce maximum settlement only, in the deepest sections of rockfill.

Perhaps the areas most vulnerable to the formation of open cracks are those in the vicinity of the high water line of the reservoir and the abutments of the dam where water pressures and rock loads are relatively light and lateral settlement is greatest. Under unfavorable and unfortunate circumstances which usually can be avoided, open cracks might develop in these regions.

Assuming identical characteristics of fill materials and conditions insofar as they are applicable to either type of dam at a given site, it would seem that movement of the fill materials caused by differential settlements would be more severe in the inclined core than they would be in a central core. Therefore, in constructing the inclined core type of dam greater care must be taken in order to obtain a dependable cutoff and downstream transition zone as well as good compaction in the Main Rockfill section. However, if the dam is well designed and constructed and the impervious zone is completely confined between the two transition sections as it is intended to be, it is the writer's opinion that the pressures exerted upon and through the moist inclined core material are sufficient to prevent formation of open cracks during the process of more or less gradual settlement of the underlying fill materials.

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The writer agrees that rockfills, particularly of large size stones, can withstand considerable flow through the fill and over pour but such should not be considered in the spillway design. However, economies in diversion and construction may be attained by taking a calculated risk of overpour during construction. In the case of Dix Dam, mentioned by the authors, flood waters rose rapidly during the construction period and submerged about 37,000 sq. ft. of uncovered placed rock and dumped fill. The difference between reservoir level and tailwater was about 36 feet. A very large volume of water passed through the dumped rock fill with no noticeable displacement of rock.

Pacific Gas and Electric Company's Upper Bear River Dam withstood, for several hours, overtopping to a maximum depth of 15 inches. The dam is 75 feet high and has an upstream face membrane on slopes of 1/2 and 3/4 to 1. The downstream face consisted of a single layer of well placed dry rubble laid on slopes of 1/2, 3/4 and 1 to 1. The fill between the upstream and downstream rubble faces consisted largely of a mixture of small rock and fines. After the flood it appeared that there had been some movement of the downstream rock facing. Subsequently the dam was reinforced by dumping rock against the downstream face and the spillway was enlarged. A currently constructed rockfill of large, well-sluiced rock would certainly withstand such overpour with a greater margin of safety.

The authors state that rockfill usually has a shear coefficient greater than 0.7. In the writer's opinion, dumped and sluiced quarried rockfill becomes so interlocked that a shear coefficient of unity can conservatively be used in design. The existence of many dams with 1/2:1, 3/4:1 and 1:1 slopes and nominal amounts of surface placed rock is substantial evidence of a high shear value.

The writer is in full accord with the statement that reasonably clean and compact gravel may be used as a foundation except under the impervious zone. Careful examination of several river bed gravel deposits has revealed essentially complete rock to rock contact through sizes ranging from large boulders to coarse sand with interstices completely filled with smaller rocks and sand particles. Such deposits surely are more compact and secure than any artificial fill that can be made to replace them.

The authors mention natural dumped rockfill slopes ranging from 1.35 to 1.40 horizontal to 1.0 vertical. The writer's experience, generally, has been with slopes of 1.30 to 1.35 horizontal to 1.0 vertical.

The 172-ft. Cogoti Dam in Chile was built to an average downstream slope of 1.6 to 1. It withstood an extremely severe earthquake with practically insignificant disturbance of the fill. The vertical settlement caused by the quake measured 1.4 feet.

The authors' comments in regard to quarry operation, dumping of rock, and the amount of permissible fines are especially commendable.

The problem of permissible fines is difficult. In writing Specifications, percentages seem to be the only means of specific control. In the field it becomes a matter of judgment, and differences of opinion between constructor and owner arise. The writer has found that a pricing schedule that causes both the constructor and the owner to lose some money on rejection of material does much to solve the problem.

The authors' statements favoring high lifts and heavy sluicing of the rockfill are well taken. Assuming suitable rock is available, these two factors are most important. High lifts are more adaptable to the inclined core type than to the vertical core type. This is a distinct advantage construction-wise and will give the minimum settlement in the main rockfill.

The behavior of the 275-ft. San Gabriel Dam No. 2, a rockfill with upstream membrane, is cited. The dam was constructed in 25-ft. lifts and sluicing was prohibited. The rock, though hard, contained some weak cleavage planes. Measurements taken during the later stages of construction indicated an accumlated total settlement near the crest of more than 15 feet in seven months. Of this amount, the last several feet of settlement came in one week of heavy rainstorms during which 12 inches of rain fell in one day. One effect of that settlement is shown in Fig. 7. An extensive, deep crack 9 inches to 14 inches wide developed near the right abutment and the crest of the dam. This is in contrast to a maximum crest settlement after two reservoir fillings of 1.13 feet (0.5% of the 240 ft. height) of Pacific Gas and Electric Company's new Bear River concrete face rockfill dam, constructed in high lifts and well sluiced.

The La Joie Storage Dam, British Columbia Electric Company, Ltd., on Bridge River is of the rockfill type with a temporary timber face overlying a 14-inch unreinforced concrete sub-slab. The deepest fill overlies the river channel at old La Joie Falls where the foundation and abutment slopes are quite steep. Here the alignment of the dam is concave to the reservoir. The dam was constructed originally to a height of about 215 feet and presently is being enlarged to its ultimate size. Differential settlement near the right abutment of the original structure caused the formation of an extensive crack in one of the timber sills. At the point of maximum separation the crack measured about 7 inches in width and there was a 3-inch downstream offset. Adequate repairs were made and little further settlement occurred.

The writer agrees with the authors' emphasis on sluicing but not entirely with the methods described by them. Fortunately, it appears that the quantity of water applied at the time and location of dumping is more important than the method of applying the water. Usually the fines tend to accumulate at the point of dump and for a short distance down the slope. Concentration of large volumes of water in these areas is more effective than high nozzle velocities. Also, there is beneficial action beneath the surface by erosion from occasional steady flow at one point. However, one should continually strive to apply water in the most effective manner.

Excellent results also have been obtained in many cases with the monitor located on the rockfill bench to one side and a short distance back from the point of dump as illustrated in Fig. 8. The stream is played diagonally into the rock as it slides off the truck and on down the rockfill face. The monitors are mounted on short skid-type carriers with a liberal length of hose. They can be speedily moved to new positions by the bulldozer which normally is in use on the rockfill bench. A long pipe handle fastened to the monitor barrel, and counter-weighted, permits easy operation of the monitor by one man.

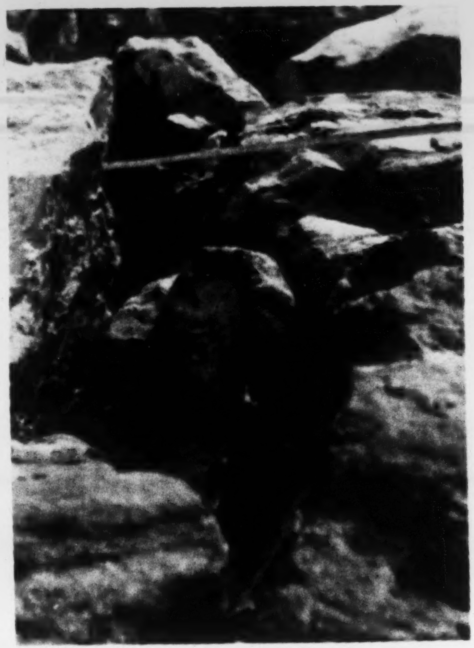

FIG. 7.—CRACK IN ROCKFILL DURING CONSTRUCTION OF SAN GABRIEL DAM NO. 2

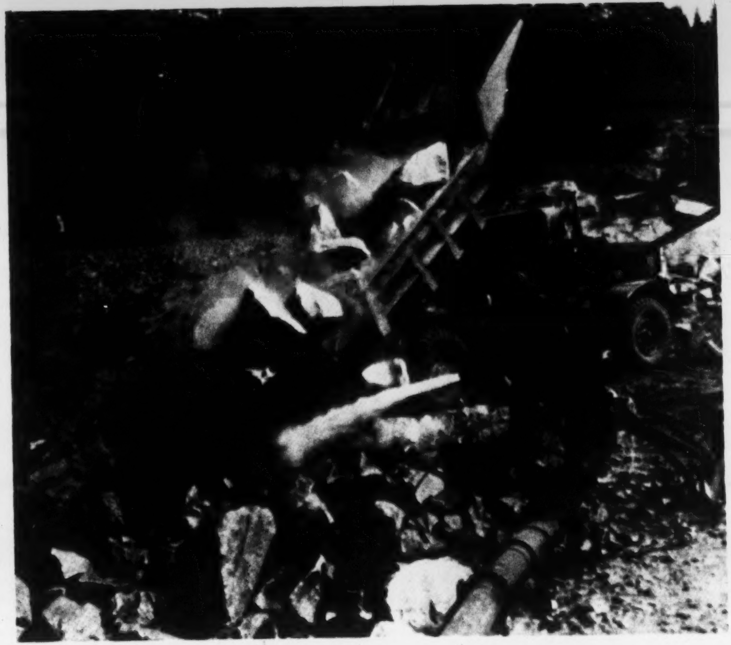

FIG. 8.—SLUICING OF DUMPED ROCKFILL WITH SKID MOUNTED, HAND OPERATED MONITOR DURING CONSTRUCTION OF LOWER BEAR RIVER DAMS.

In conclusion, the writer desires to state that he is in agreement with the authors that a certain amount of fines, with sluicing, acts as a lubricant. Slides frequently occur where water is applied in quantity, especially to areas containing local concentrations of fines, and do permit better adjustment of individual rocks. However, he prefers a minimum amount of fines and would not advocate deliberate inclusion of fines in order to accommodate such action.

D. J. BLEIFUSS,[5] F. ASCE AND JAMES P. HAWKE,[6] F. ASCE.—The writers are very appreciative of the comments of Mr. Steele. Mr. Steele has been responsible for the design and construction of many important dams, a number of which are rockfill dams of the type he designates as true rockfill dams (comprising a rockfill with an upstream face membrane of concrete, timber or gunite) and comments from his experience are most authorative.

Mr. Steele very properly emphasizes the importance of constructing rockfill dams so as to minimize the differential settlement throughout the dam. This becomes of particular importance in connection with many of the higher rockfill dams now (1960) under construction. Mr. Steele's comments regarding the172-ft high Cogoti Dam in Chile, built with an average downstream slope 1.6 to 1 which withstood an extremely severe earthquake, is of special importance in our consideration of the earthquake resistance of high rockfill dams. The experience which Mr. Steele cites in connection with San Gabriel Dam No. 2, California and the La Joie Storage Dam, British Columbia, emphasizes the importance of his comments regarding care in selection of materials together with proper deisgn and construction methods.

During the period which has elapsed since the writing of this paper, there has been a trend toward the design of increasingly higher rockfill dams. In construction of some of these rockfill dams, it has been necessary to use available earth embankment and rockfill materials which, in some cases, were not ideal. In other instances rockfill dams have been constructed on foundations of granular materials which involved special problems in foundation stability affecting the design of the dam. Accompanying these factors has been the use of steeper outer slopes on the upstream and downstream shells of certain rockfill dams.

The comprehensive effect of the foregoing factors has been to make design problems more exacting. To adequately cope with these design factors we have an urgent need to know more about the actual shear strength of cohesionless material, both rockfill and gravel fill, under the high vertical and lateral pressures existing in dams over 400-ft high. Some laboratory work has been performed which would indicate that there may be a reduction in the shear strength of granual materials under such loading conditions.

The earth materials used in the core and the materials used in the filters become more critical for high dams. It is desirable to re-examine the problem of construction pore pressures and consider carefully how closely our laboratory triaxial shear tests approximate the pore pressure conditions existing in the core of a high rockfill dam in the period prior to and during the initial filling of the reservoir.

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From the standpoint of rigorous structural analysis our present methods of stability analysis are open to question. The assumption of failure along a thin slice of unity width introduces approximations, particularly in the case of steeply sloping abutments which may be overly conservative. This would indicate a need for more realistic methods of stability analysis based on the various modes of failures which might rationally be expected.

Certain high rockfill dams involve diversion and closure problems which require the closure section of the dam be placed in a single dry season. This leads to a need to make the end slopes of the initial stage of the embankment as steep as practicable in order to minimize the amount of material to be placed in the closure section. Studies of such end slopes by the method of thin slices leads to unrealistically flat slopes since the unit width section may lie entirely within the impervious core. Obviously, however, any large scale failure of the end slope would involve the adjacent cohesionsless shells, and the effects of the shells should be considered in the stability analysis. This is being done on recent projects with which the writer is connected and it is hoped that further study along this line may lead to more realistic and satisfactory methods of stability analysis of composite earth and rockfill dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3073

ROCKFILL DAMS: KAJAKAI CENTRAL CORE DAM, AFGHANISTAN

By Glenn F. Sudman,[1] F. ASCE

With Discussion by Messrs. F. L. Lawton; and Glenn F. Sudman

SYNOPSIS

This paper describes the design and construction of the rockfill dam embankment portion of Kajakai Project. This dam, in Central Afghanistan, was economically constructed despite its geographic isolation. The feat required maximum use of locally obtainable materials and complete training of a native work force.

INTRODUCTION

The Royal Government of Afghanistan has initiated an improvement program for revitalizing an area which, in past history, was referred to as the "Garden Spot" of Asia. This area, which in modern times is called "Helmand River Basin", occupies approximately 107,000 square miles of Southern Afghanistan. The major lifeline of the basin is the Helmand River. As shown on Figure 1, the river flows in a 600-mile circuitous path from the Hindu Khush spur of the Himalaya mountains to the Seistan Sink on the Iranian border.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1735. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The precipitous, barren slopes of the upper basin, combined with a climate predominated by winter rainfall and dry hot summers, have resulted in extreme annual variations of river flow. The fertile valley lands have been subjected to alternate periods of drought and flooding. In order to alleviate this serious condition, the combined forces of International Engineering Company, Inc. and Morrison-Knudsen Afghanistan, Inc. were commissioned to design and construct a dam and reservoir for control and development of the Helmand River.

Reconnaissance of the river revealed the best damsite was near the village of Kajakai, about 75 air miles northwest of Kandahar. At this site, the ancient Helmand River had cut through beds of limestone to form a short, steep-walled gorge. Upstream from the gorge, the river valley widens and is encompassed by low rolling hills. The natural topography thus provided an excellent site where a considerable volume of reservoir storage could be economically developed.

Preliminary investigations of streamflow data, irrigation demands, downstream flow requirements, and power development potentialities resulted in the selection of a multi-purpose project. Through economic analyses led to the decision that the most feasible project development would be attained by construction in two stages.

The first stage, as shown on Figure 2, included a main embankment, side saddle spillway, and two diversion tunnels. Various combinations of dam and spillway arrangements were studied. These resulted in the selection of a dam as shown on Figure 3, with top at Elevation 1050 meters. The spillway crest was set at Elevation 1033.5 meters. Spillway discharges were uncontrolled. Reservoir capacity to the top of the spillway crest would then be approximately 1,470,000 acre-feet. This amount was considered adequate for water requirements, mostly irrigation, under the present scope of land development.

After diversion use, one of the tunnels would be fitted for irrigation releases. The other tunnel would be plugged until power requirements justified its further development.

The second stage can be constructed when irrigation demands increase to where more water is required, or when a market for power develops. This stage will consist primarily of the installation of gates in the spillway channel and construction of the remaining power facilities. Allowing five meters of freeboard on the dam crest would set the top of the gates at Elevation 1045 meters, and would provide an additional 830,000 acre-feet of storage, making the total reservoir capacity approximately 2,300,000 acre-feet. This amount would be adequate for irrigating almost 500,000 acres, an area slightly greater than any land development project contemplated in the foreseeable future. In addition, irrigation releases and the developed head would provide energy for a potential firm power load of 60,000 kilowatts. Selection of the actual type and size of generating units has been deferred until the need for power is established.

7

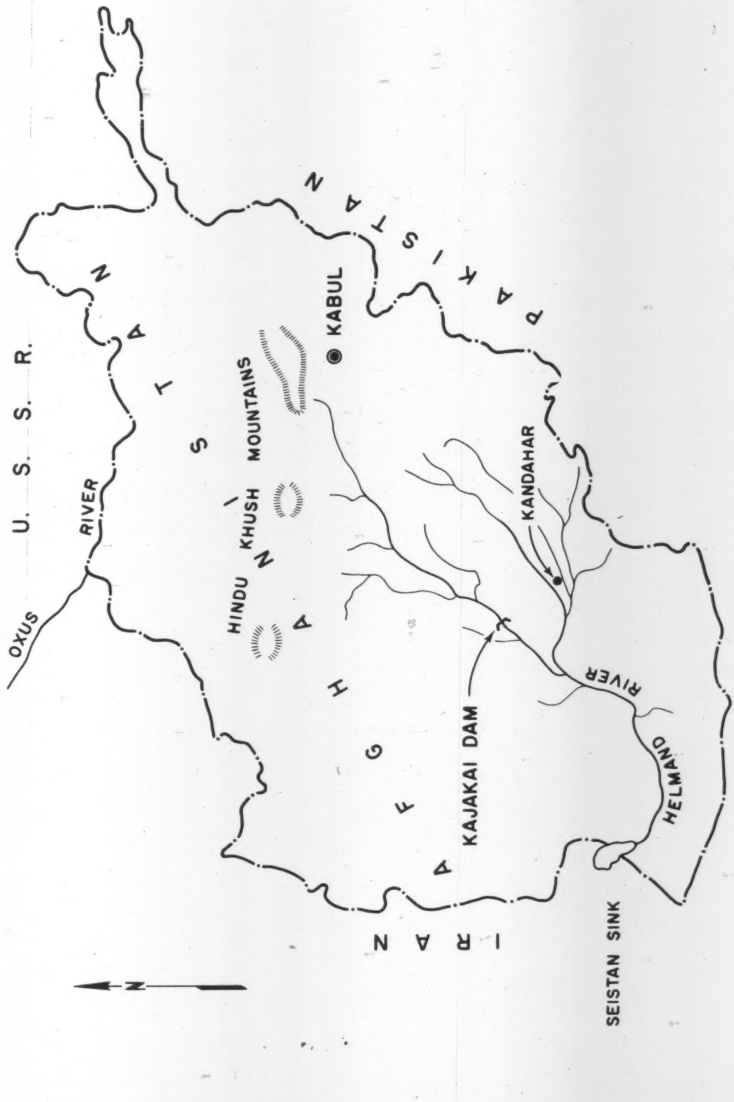

U. S. S. R.
OXUS
RIVER
AFGHANISTAN
HINDU KHUSH MOUNTAINS
KABUL
PAKISTAN
KANDAHAR
KAJAKAI DAM
HELMAND
RIVER
IRAN
SEISTAN SINK
N

FIG. 1.—LOCATION MAP FOR KAJAKAI DAM

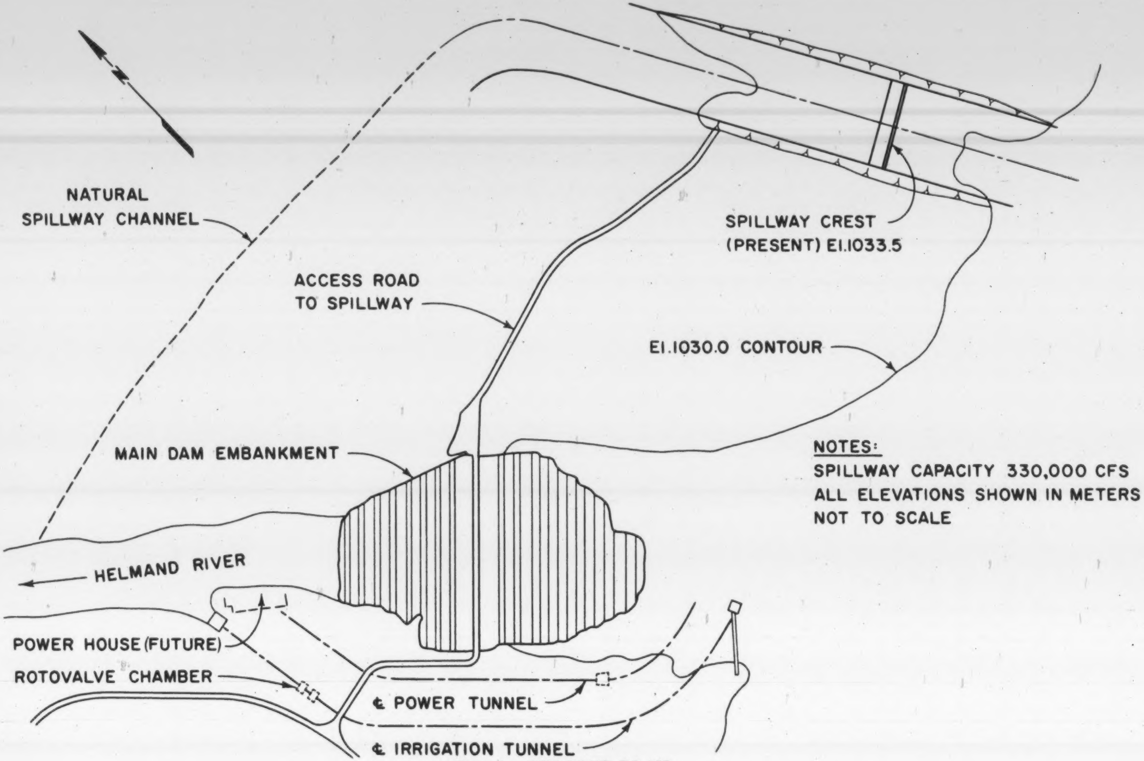
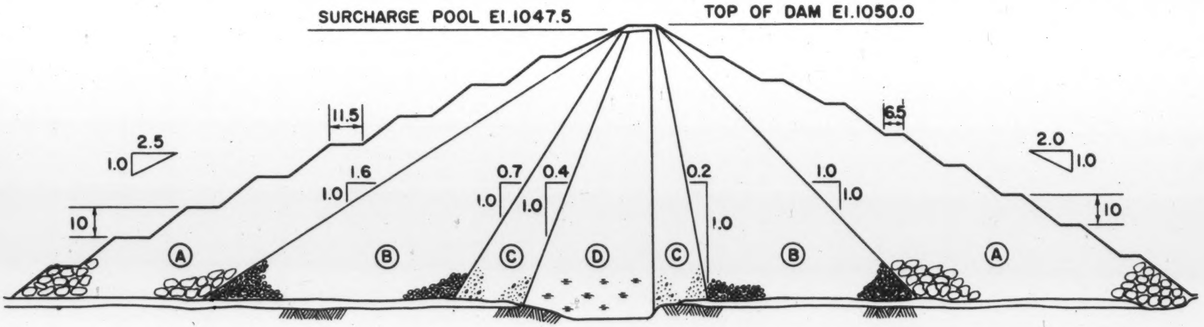

N
NATURAL SPILLWAY CHANNEL
SPILLWAY CREST (PRESENT) El.1033.5
ACCESS ROAD TO SPILLWAY
El.1030.0 CONTOUR
MAIN DAM EMBANKMENT
NOTES:
SPILLWAY CAPACITY 330,000 CFS
ALL ELEVATIONS SHOWN IN METERS
NOT TO SCALE
HELMAND RIVER
POWER HOUSE (FUTURE)
ROTOVALVE CHAMBER
℄ POWER TUNNEL
℄ IRRIGATION TUNNEL

FIG. 2.—PROJECT PLAN


SURCHARGE POOL El.1047.5
TOP OF DAM El.1050.0
11.5
6.5
2.5
1.0
2.0
1.0
1.6
1.0
0.7
1.0
0.4
1.0
0.2
1.0
1.0
1.0
10
10
A
B
C
D
C
B
A

LEGEND
Ⓐ ROCK
Ⓑ FREE-DRAINING GRAVEL
Ⓒ TRANSITION ZONE
Ⓓ IMPERVIOUS

NOTES:
TOTAL EMBANKMENT VOLUME 4,230,000 CU. YD.
ALL DIMENSIONS AND ELEVATIONS SHOWN IN METERS
NOT TO SCALE

RESERVOIR STORAGE CAPACITY

DEAD	(EL.1008.0)	440,000 AC FT
INITIAL	(EL.1033.5)	1,470,000 AC FT
FUTURE	(EL.1045.0)	2,310,000 AC FT

FIG. 3.—EMBANKMENT SECTION

Investigations

Geological examination of the damsite and reservoir area was conducted during the summer of 1950. In general, rock at the site was found to be massive dolomitic limestone. Beddings consist of thick layers of limestone near river level, becoming thinner with a few cherty members and occasional shale interbeds near the tops of the abutments. The beds dip generally about 6 to 8 degrees toward the northwest. Several faults were visible but were not considered serious.

Extensive jointing of the rock was visible at the abutments. Joints were in two directions intersecting each other at right angles, with the pattern forming oblique angles to the axis of the dam. The joints varied from close to open and were more or less filled with clay. Surface caverns were disclosed on some of the principal joints.

Foundation borings consisted of 23 drill holes in the river channel and one horizontal hole in the left abutment. Overburden in the river channel consisted of about 30 feet of sand, gravel, and boulders overlying sound limestone bedrock. The hole in the left abutment near the axis of the dam was driven 210 feet and revealed no apparent change in structure of the bedrock.

Investigations of potential construction materials were conducted near the site during 1948 and 1949, and were supplemented by laboratory tests made in the United States. Test pits were dug at about 100-meter intervals in three areas. Mechanical analyses and compaction tests were performed on materials from these test pits.

Ample quantities of materials suitable for the construction of a rockfill dam were found in two borrow-pit areas located from one to two kilometers from the dam, one upstream and the other downstream from the gorge.

Design

Discussion of design of the features completed under the first stage of construction is limited in this paper to the main dam embankment. The other features of the dam are discussed only as they effect design or construction of the main embankment.

Selection of Type of Dam

A rockfill type of dam was selected for development of the Kajakai site. Design of rockfill embankment was predicated on the efficient use of economically available construction materials and equipment. Preliminary investigations had shown that an adequate supply of suitable impervious and pervious materials were available nearby. The limestone rock from required excavations was found to be entirely suitable for incorporation in the dam embankment. In addition, construction plant and personnel were available in this remote country, having been assembled for movement of earth and rock on a large scale for other projects in the Helmand Valley.

Conversely, cement and concrete plant equipment and personnel would have to be imported. Supply lines to Afghanistan are complicated routes and require several re-handlings of all materials. Each link of the supply line is subject to interruptions, which in turn interrupt construction schedules dependent upon outside supplies. The long line and interruptions result in high construction cost of any structure requiring large amounts of imported bulk items such as cement.

Diversion

The diversion scheme was based on construction of a low embankment upstream from the dam, of sufficient height to divert low water flow through one of the two diversion tunnels. During the eight-month period before high runoff could be expected, the downstream portion of the rockfill section was to be raised to Elevation 985 meters. At the start of the high water season, both tunnels will be available for diversion. Water at Elevation 985 meters would provide enough head to pass the maximum diversion design flood of 70,000 cfs through the tunnels. Two 32-foot diameter concrete-lined horseshoe-shaped tunnels would have the required capacity and would provide flexibility of use during construction. This was especially true for completion of the work within the separate bores and for adaptability to final water use.

If the diversion design flood was exceeded, minor overtopping of the downstream rockfill section could occur without damage, as a protective coating of heavy rock was specified. In addition, the embankment upstream of this rockfill was to follow at a lower level to obtain smaller velocities over the portion of the dam having finer grained materials. This was in agreement with construction schedules as the time required for foundation clean-up and grouting in the impervious area precluded placement of a large amount of material during the first eight-month period.

The target estimate type of contract permitted final designs to be easily and quickly adjusted as conditions were disclosed during construction. The jointed condition of the rock had led to the conclusion in the original design that the tunnels would require lining throughout. After the tunnels were started, it became apparent that the rock was much better than anticipated. Consequently, the design was immediately modified to eliminate the lining without the delay frequently occasioned by contract re-negotiations.

The nominal unlined tunnel size was changed to 34 feet. At the same time, review of construction operation schedules revealed that the top of the downstream rockfill could be constructed to Elevation 990 meters. The diversion capacity resulting from these modifications was computed to be 63,000 cfs.

Spillway

The spillway arrangement was based on: advantageous use of the topography to avoid paving of the spillway outlet channel, and to balance the amount of spillway excavation as nearly as feasible with the amount required in the embankment.

Rock in the selected outlet channel is adequate to withstand the erosive forces of water for many years. Some lining may be eventually required as a maintenance feature. The spillway outlet channel excavation terminates in a natural ravine leading to the river at a point about 300 meters downstream from the toe of the dam.

Two hypothetical floods were used for spillway design. One flood with a peak of 318,000 cfs was to be routed through the reservoir without encroaching on the freeboard. The other flood with 50 percent greater magnitude was not to overtop the dam.

The first-stage, ungated spillway has a capacity of 330,000 cfs and a usable surcharge of more than 1,000,000 acre-feet at normal freeboard level. A check review of hydrological information accumulated in the intervening period will be made to verify the accuracy of the design floods at the time the gates are added.

Main Embankment

The main embankment, as shown on Figure 3, has a crest length of 275 meters and is approximately 100 meters high as measured from bedrock to top of dam. The total embankment volume is approximately 3,200,000 cubic meters.

Complete removal of overburden material was required only in the impervious core area. In the remaining embankment area, removal of only unstable materials containing clay or silt was required. Naturally-deposited river sand and gravel, which was left in place, was to be scarified to a depth of six inches and re-compacted prior to placing the embankment.

On the abutments, bedrock was exposed over much of the area (Figs. 4 and 5). Consequently, treatment consisted of removal of overhangs, sloping of vertical faces and elimination of sudden changes in slope. This abutment sloping was required to provide a wedging action as the fill consolidated, and to eliminate the possibility of the fill "Hanging up" and cracking at the abutment contact during the consolidation process.

The zoning within the dam was predicated on the most efficient use of the available materials. Preliminary investigations had indicated that definite quantities of materials, varying from impervious to previous, were available. From the tunnel and spillway designs, estimates were made of the quantities of rock available from these excavations. An early decision was to keep the core and transition zones at a minimum thickness consistent with prevention of percolation through the dam and the efficient use of placing and compaction equipment. Several factors influenced this decision: relative availability of materials, cost-in-place of the fill, and the comparative lack of structural strength of these types of material. Further decisions limited the use of rock to that available from required excavation. Quarried rock would be more costly for an equivalent effect on dam stability than pervious materials from borrow pits. As the actual quantity of suitable rock available from required excavations would probably differ from the estimated amount, zoning limits between gravel and rock materials were made flexible.

As finally designed, the dam consisted of:

a. An impervious core of compacted material composed of a mixture of alluvial sand, silt, and clay.
b. Zones of compacted transition material upstream and downstream of the impervious core.
c. Pervious sections of compacted free-draining gravel upstream and downstream.
d. Rockfill sections, upstream and downstream, comprising the outer shells of the dam.

The embankment slopes were determined by stability analyses, using conservative strength factors for the various materials. The factors of safety, as determined from these analyses, under all conditions of loading, exceeded the minimum values generally considered permissible.

Embankment Materials

Specifications

Specifications for field control were issued to ensure that the minimum requirements utilized in the stability analysis were satisfied.

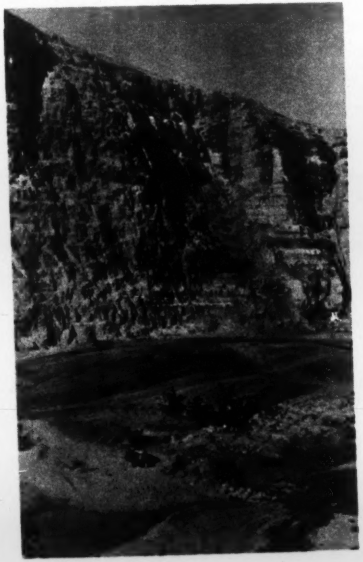


FIG. 4.—KAJAKAI DAM, EXCAVATION OF CORE TRENCH, START OF IMPERVIOUS CORE FILL, AND STRIPPED LEFT ABUTMENT

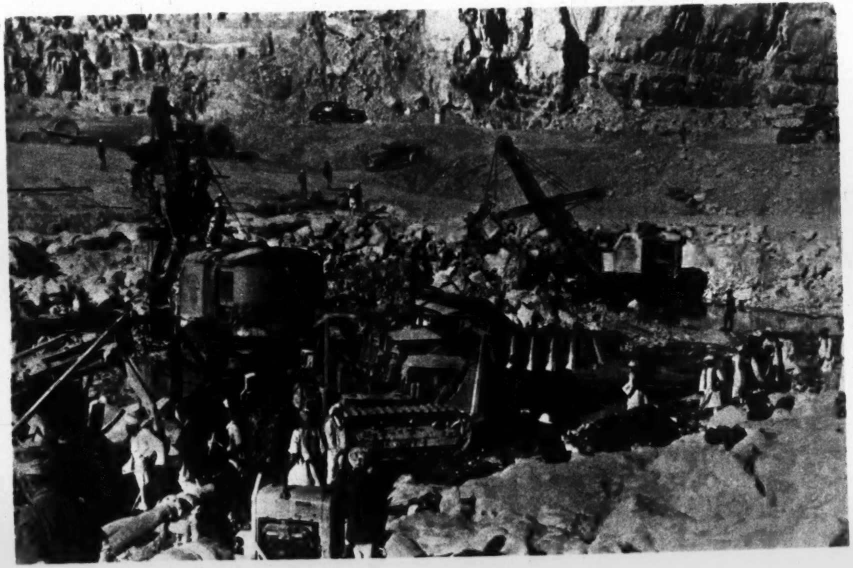


FIG. 5.—KAJAKAI DAM, EXCAVATION OF CORE TRENCH, LOOKING TOWARDS LEFT ABUTMENT

The impervious material was specified to be a well-graded mixture of sand, gravel, clay, and silt possessing low to medium plasticity with not more than 40 percent gravel and no stones larger than 4 inches in diameter. The permeability after compaction was not to exceed 1.0 x 10^{-4} centimeters per second (103.5 feet per year). Minimum shear strengths, as determined from triaxial shear tests of saturated, unconsolidated, undrained representative samples, were stipulated to be equivalent to an angle of internal friction of 17 degrees and an apparent cohesion of 1.5 kilograms per square centimeter. Compaction was to be 98 percent of standard density (ASTM Test D-698), with a moisture content of between two and four percent on the dry side of optimum.

Grading of the transition zones from impervious on the inside to pervious on the outside, with shear strength equal to or greater than the impervious zones, was specified. Special selection of material was required for a 20-meter width adjacent to each abutment, which was to be highly impervious, and for a layer of free-draining, highly pervious material overlying the foundation in the downstream portion of the dam.

The gravel zone material was specified to be free-draining with a permeability after compaction of not less than 400 x 10^{-4} centimeters per second (41,413 feet per year). Compaction was required to a dry density of 95 percent of the maximum dry density obtained in a laboratory test. An angle of internal friction of 34 degrees was specified to agree with the value utilized in design.

The rockfill was to consist of sound limestone obtained from excavations, sluiced into place with the volume of water twice the volume of rock. Special selection of large, more resistant rock was required on the downstream face below Elevation 990 meters and on the upstream face between Elevation 1010 meters and Elevation 1050 meters. Lifts were specified to be dumped in 10-meter heights with outside slopes equal to the natural angle of repose. Berms of appropriate widths were to be provided at the top of each lift to give average slopes equal to the design slope. Variation in the slope of the contacts between gravel and rockfill zones was stipulated to suit the quantity of suitable rockfill material found available from the required excavations.

Field Control

No unusual problems were encountered in the placing and control of materials. The borrow pits were found to be characterized by a gradual transition from fine impervious silts to coarse river-run gravels, with no clearly defined lines of demarcation. Careful field control using permeability limits as a guide was exercised so that permeability in the transition zone between the impermeable core and the pervious gravel shell was increased gradually. Tests of the in-place materials showed that shear strengths greatly exceeded minimum requirements. The relative ratio of permeabilities was also substantially higher than required. The moisture content of the materials in the borrow pit was very low. However, this was economically corrected by diking borrow areas and flooding them until the moisture content was satisfactory. Compaction to the specified densities was readily accomplished by sheepsfoot rollers and by pneumatic hand tampers.

The advantage of providing flexibility in the design of the embankment was demonstrated during construction. The faulted zone in the spillway proved to be larger than anticipated, with the result that the amount of rock suitable for use in the embankment was less than originally estimated. As construction of the embankment proceeded, the substitution in the upper portion of the dam of large gravel and cobbles in lieu of a portion of the exacavated rock was approved after re-examination of stability.

A second example of flexibility was in the use of impervious materials from borrow areas. The placing of impervious material in the embankment was delayed by fault treatment in the core trench area. Because of the delay, the amount of materials used from the upstream borrow pit was less than originally scheduled. As a consequence, more extensive use of the downstream borrow area was required. This possibility had been recognized and the existence of an adequate reserve had been determined beforehand.

Foundation Treatment

The pattern of intersecting joints, solution channels and faults crossing the site necessitated extensive and careful treatment to ensure that leakage through the rock would be minimized. A very careful study in the field, accompanied by surveys of the main open joints and water testing through drilled holes, resulted in the adoption of a final program consisting of three general operations (Fig. 6).

Shafts were excavated in the faulted areas for satisfactory cutoff and backfilled with concrete. Surface exposures of joints which would be subjected to water pressure were cleaned out and filled with gunite or concrete plugs. Holes were then drilled diagonally to intercept the greatest number of joints and then grouted.

The joints in some places were found to be filled with clay which would tend to prevent the flow of grout, but which in time might be removed by flow of water. For this reason, it was felt that the sealing of surface exposures that might otherwise permit the upstream entry of water into the pattern of intersecting joints was of prime importance. Due to the directional pattern of the jointing system, special attention was directed to the left abutment where the treatment of surface exposures was continued for some distance upstream. The segments of the tunnels which would be subjected to reservoir pressure were similarly treated. In some locations on the left abutment and on the tunnel walls, the gunite seal blanketed extensive areas of exposed rock surface. Where grouting was performed in the vicinity of capped exposures, the surface treatment prevented the escape of grout except through vents left specifically for that purpose.

The foundation excavation disclosed three nearly vertical faults cutting diagonally across the core; one in the right abutment, one near the toe of the right abutment, and the other near the middle of the river.

The right abutment faults were filled with tight clay and gouge. These faults were four and six feet wide and were excavated to about 70 feet and backfilled with concrete.

Treatment of the fault in the center of the river became a major operation. The faulted zone was about 16 feet wide and filled with fractured but solid rock, interspersed with bands of dense calcite laminated with cement gouge and clay. The shaft, keyed into solid rock was located near the upper side of the core and had a length slightly greater than the width of the zone. Excavation of the shaft in this fault required blasting. However upon exposure after blasting, the clay slaked and shrank, loosening the walls causing cave-ins and rockfalls.

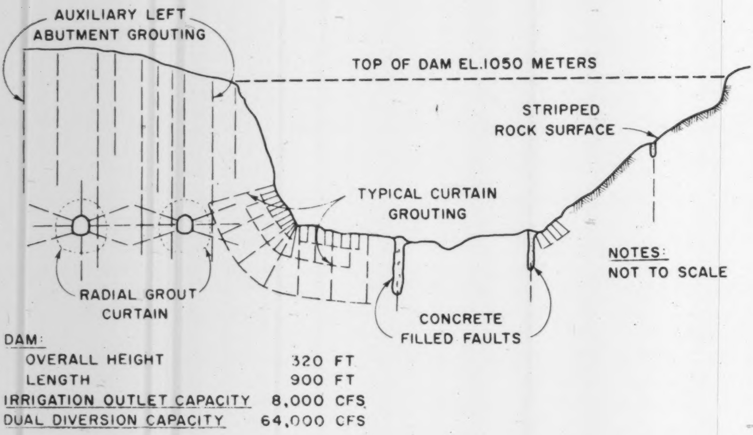

AUXILIARY LEFT ABUTMENT GROUTING
TOP OF DAM EL.1050 METERS
STRIPPED ROCK SURFACE
TYPICAL CURTAIN GROUTING
NOTES: NOT TO SCALE
RADIAL GROUT CURTAIN
CONCRETE FILLED FAULTS
DAM:
OVERALL HEIGHT 320 FT
LENGTH 900 FT
IRRIGATION OUTLET CAPACITY 8,000 CFS
DUAL DIVERSION CAPACITY 64,000 CFS

FIG. 6.—LONGITUDINAL SECTION

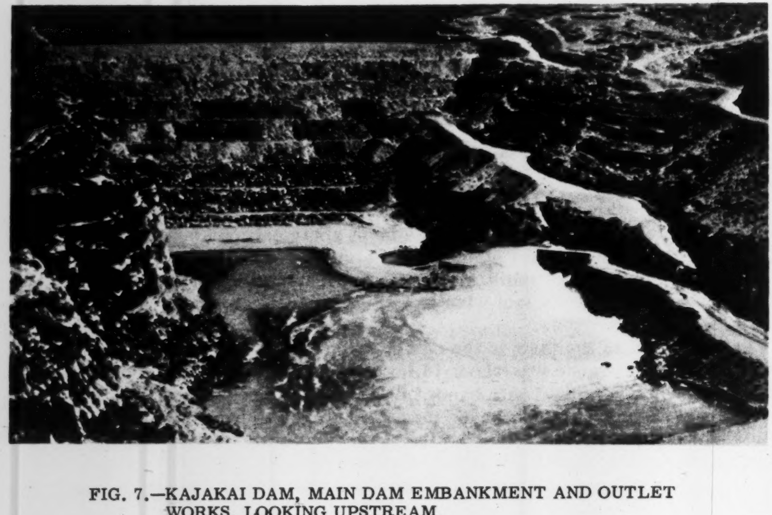


FIG. 7.—KAJAKAI DAM, MAIN DAM EMBANKMENT AND OUTLET WORKS, LOOKING UPSTREAM

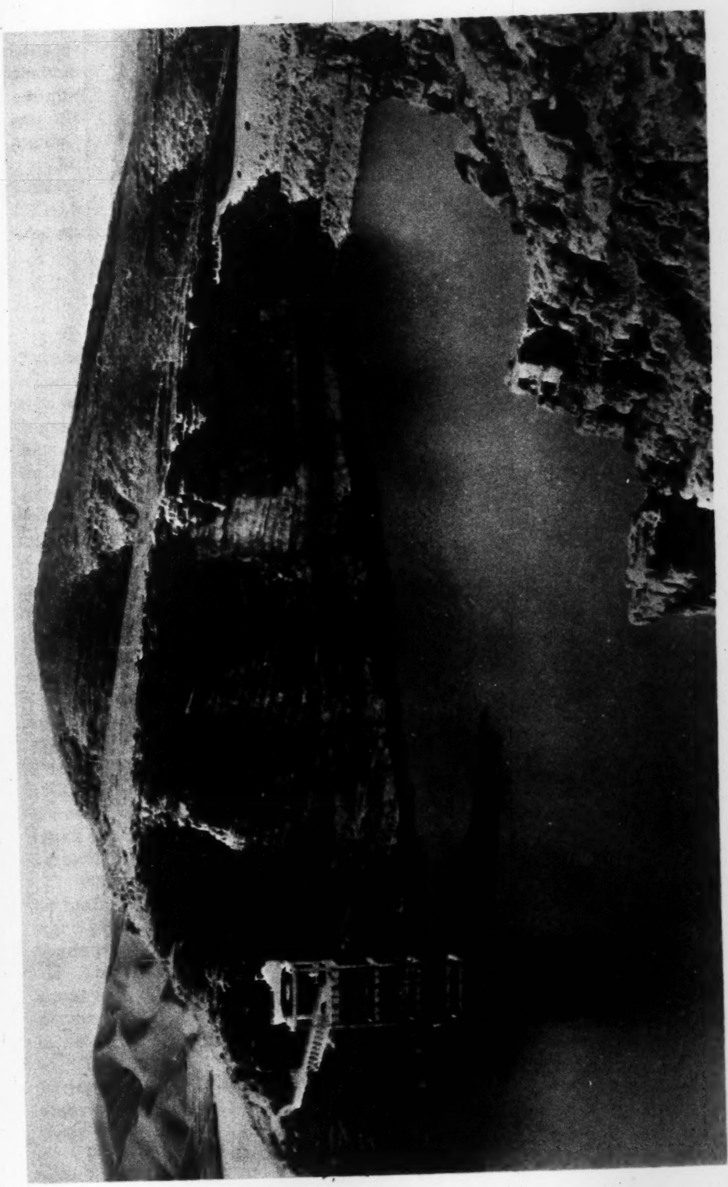


FIG. 8.—KAJAKAI DAM, MAIN DAM EMBANKMENT AND IRRIGATION INTAKE TOWER, LOOKING DOWNSTREAM

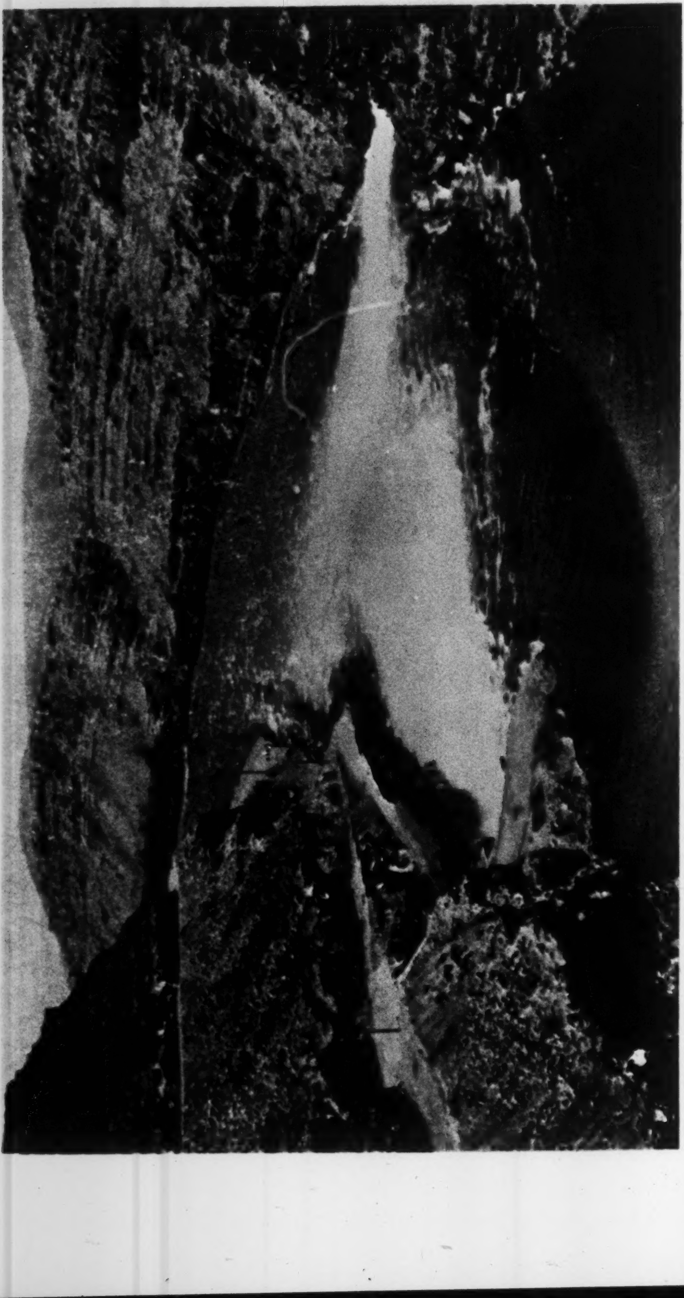


FIG. 9.—KAJAKAI DAM, COMBINED DISCHARGE OF SPILLWAY AND OUTLET WORKS, LOOKING DOWNSTREAM FROM TOP OF DAM

The excavation of this shaft proceeded very slowly and proved to be a construction bottleneck. After the shaft reached a depth of 60 feet, work became very hazardous. The upper part was then plugged with concrete. An access opening was left through the plug to the underlying excavation. After completion of the excavation to a depth of 100 feet, the entire shaft was plugged with concrete. The total amount of concrete required was 1,800 cubic yards. A grout seal was made between the plug and the rock.

In addition to the main shaft in the fault, the surface of the zone was cleaned out to a depth of five feet for its entire length across the core and backfilled with concrete. A second shaft at the downstream edge of the core zone was excavated to a depth of 15 feet and backfilled with concrete.

Construction

It might be expected that the construction of a major project such as Kajakai Dam in the historically isolated country of Afghanistan would prove to be unusually difficult. This however, was not the case, as the construction was accomplished essentially as scheduled and with no unexpected problems resulting from the isolated location. A construction organization had been operative in Afghanistan for about three years prior to the actual start on Kajakai Dam. In this period of time, established procedures had been developed for maintaining supplies of construction materials, equipment, and spare parts. These included well-planned requisition procedures employed by an efficient procurement organization, and a skilled traffic department.

The only construction materials available locally were earth and rock, except for a small amount of poor grade lumber. Everything else had to be imported. Except for cement from Belgium, and fuel and lubricants from the Middle East, imports were from the United States. Food for the American personnel was imported except for meat, fresh vegetables, and fresh fruit in season.

All imported material was received at the Port of Karachi, West Pakistan, where the Constructor maintained an organization to facilitate trans-shipment. The next step was by railroad to Chaman on the Pakistan-Afghanistan border. The supplies were then carried by truck to the main camp and service center at Kandahar over a 70-kilometer highway constructed as one of the first phases of development work in Afghanistan. From Kandahar to the damsite, truck transport was over a 170-kilometer road, a portion of which was built as part of the Kajakai Project.

Storage and service facilities at the site were comparable to what might be maintained at similar projects in the United States. Back-up facilities were provided at the main center in Kandahar, which had been developed as headquarters for most of the work in Afghanistan.

The construction camp at Kajakai included housing and mess facilities, equal to living standards in the United States, for American and other foreign personnel. The buildings were generally of stone construction, which proved to be satisfactory for this locality.

Facilities required by the native workmen were constructed by themselves in separate compounds. These people were issued rations of whole wheat and rice as well as other locally produced foods from which they prepared their own meals.

Prior to and during the construction of Kajakai Dam, a large number of native workmen received training in the operation and maintenance of construction equipment, and in other contruction skills. They proved very apt pupils and no difficulty was encountered in developing an adequate number of skilled workmen.

The majority of the foreign personnel on this project were in administrative, technical, and supervisory categories. At the height of construction activity, the force at the jobsite consisted of 1850 native workmen and 69 foreigners, of whom 54 were Americans.

In May 1950, facing-off of the diversion tunnels was started. By August 1951, the low initial cofferdam fill was completed and low season flow diverted through one of the tunnels. At the end of September, the site had been unwatered, most of the overburden excavated in the core trench, the foundation treatment started, and some progress made on construction of the downstream rock section.

Success of the diversion scheme required that the planned construction schedule placing rates be carefully followed during the first phase of the work. The construction operations proved to be well planned and adequate equipment was available to meet this schedule. No difficulty was encountered except for the delay occasioned by the excavation of the shaft in the fault zone under the core trench. This fortunately caused no interference with the most critical item, the placing of the downstream rockfill. After the extent of this fault zone treatment was determined and the effect on the construction schedule realized, a wall was constructed around the area to permit the embankment placing to proceed.

Rock placement was concentrated on the downstream rockfill to ensure completion to Elevation 990 meters prior to the spring floods. This elevation was reached in March 1952, and flooding occurred at the end of the same month. The peak flow was 29,000 cfs and the water surface upstream from the fill reached a maximum elevation of 970 meters.

The dam embankment was completed in late 1952, with an average monthly placement of 390,000 cubic yards of material during the height of activity. Figures 7 and 8 show the completed embankment, looking upstream and downstream, respectively.

Performance

Spillway

Storage of water commenced immediately following substantial completion of construction. The reservoir was filled nearly to spillway crest the first season, even though continuous irrigation releases were made. Flow over the spillway occurred for the first time in 1954, with a peak flow of 24,000 cfs. Maximum spillway flow recorded to date was in 1957, when a peak of approximately 60,000 cfs occurred. Performance of the spillway during these flows has been entirely satisfactory. Overburden was left in the natural spillway channel downstream from the crest, and the expected erosion of this material has occurred with no detrimental effects. Discharge from the natural channel reenters the Helmand River downstream from the toe of the dam, nearly opposite the irrigation outlet works. The combined flow from the spillway channel and outlet works is shown on Figure 9. As shown in the picture, waves formed by the combined flow are almost completely dissipated by the time they reach the toe of the dam.

Effectiveness of Seepage Control

Effectiveness of the seepage control was noted by visual observation of the embankment. The fault treatment, grouting, and core compaction method have apparently been very successful. In addition to visual observation, piezometers installed during the embankment construction have been read at frequent intervals since the original filling of the dam in 1953, and the readings carefully studied. Pore pressures and phreatic lines derived from the readings have been plotted and are presented on Figs. 10 and 11. In general, the readings indicate that pore pressures within the impervious core follow the rise and fall in reservoir water surface elevation. There was no appreciable lag in response of pore pressure to pool variations. In all cases, even for the example taken two months after completion of the core, the phreatic line was fully developed to the stage expected for the steady state condition.

Settlement

Settlement and horizontal displacement was observed from readings of the triangulated range established at Kajakai. Between June 1954 and December 1957, the average settlement of the crest was approximately two inches, while the horizontal displacement averaged three-fourths of an inch in a downstream direction. Both figures are considered to be indicative of negligible movement.

Cost-Benefit

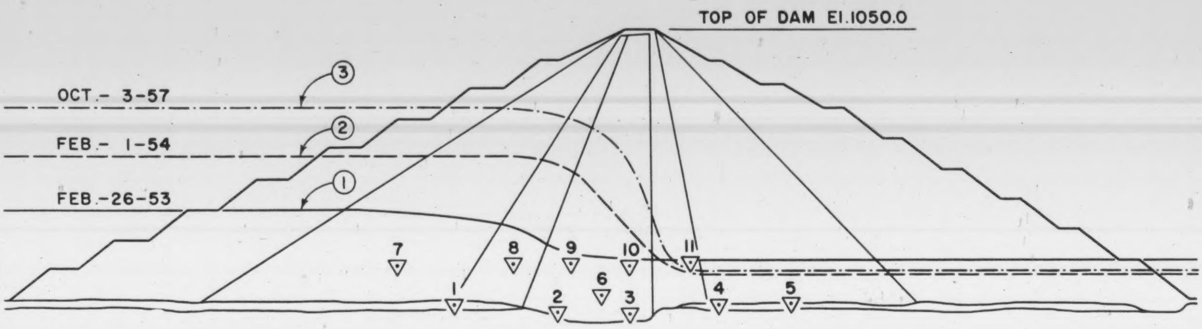
It would be very difficult to place an exact monetary value on the actual benefits derived from construction of Kajakai Dam and Reservoir. A few construction costs as related to various project features are presented, however, as an aid for individual evaluation of potential benefits.

The actual final cost of the dam approached $12,500.000. The total quantity of earth and rock in the main embankment was about 4,200,000 cubic yards; therefore, the overall cost including all structures and appurtenant work constructed in the first stage was about three dollars per cubic yard. Placing the actual earth and rockfill portions of the embankment had a final cost of about $1,000,000, resulting in a unit cost of about 25 cents per cubic yard. Based on the actual final cost, and a reservoir capacity of 1,470,000 acre-feet, the first-stage cost was about $8.50 per acre-foot.

The cost of the dam, with respect to irrigable lands, can be evaluated on the basis of land development studies. As of 1957, these studies indicated that approximately 220,000 acres were being irrigated by Helmand River waters downstream from Kajakai Dam. The annual interest and amortization of the expenditures for Kajakai Dam applied to the presently irrigated lands amounts to less than 50 cents per acre-foot.

The regulation provided by Kajakai Dam has not only enabled new acreage to be brought under irrigation, but has given a firmer and more usable supply of water to lands previously irrigated. When the lands below the reservoir are developed to the extent that full use is made of the water stored in the first stage of Kajakai Reservoir, the cost of the dam will result in an annual average cost of 36 cents per acre-foot of water.

These figures do not include the power potential at Kajakai which may eventually have an ultimate installed capacity of 120,000 kilowatts and flood


TOP OF DAM El.1050.0
OCT.- 3-57
FEB.- 1-54
FEB.-26-53
③
②
①
7
8
9
10
11
1
2
6
3
4
5

LEGEND
① POOL EL. 990, T.W. EL.970
② POOL EL.1008, T.W. EL.965
③ POOL EL.1024, T.W. EL.967

NOTES:
▽2 PIEZOMETER NUMBER
ALL ELEVATIONS SHOWN IN METERS
EMBANKMENT COMPLETED JAN.1953
NOT TO SCALE

FIG. 10.—PHREATIC LINES

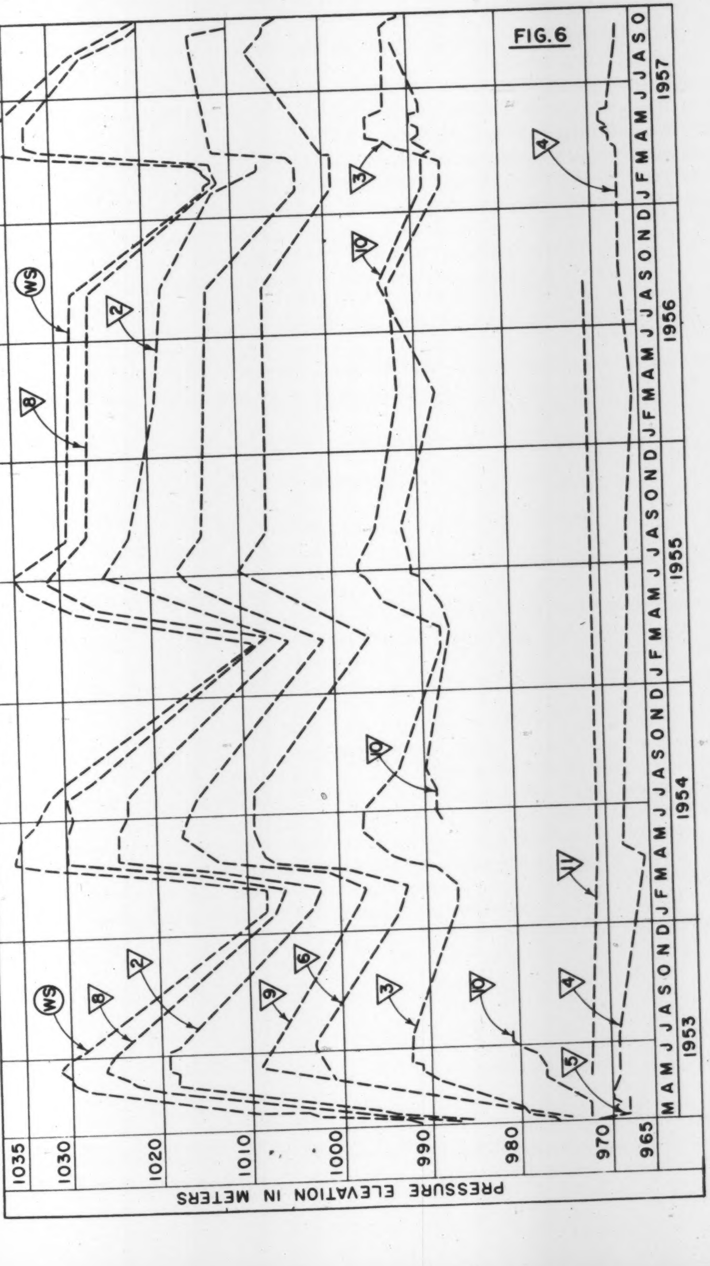


FIG. 11.—EMBANKMENT PORE PRESSURES

control benefits which are considerable. These, as well as further water use benefits, can be attained by construction of the second stage at a relatively low development cost.

Personnel

The preliminary investigations, design and supervision of construction were performed by personnel of the International Engineering Company, Inc. Construction was accomplished by the forces of Morrison-Knudsen Afghanistan, Inc. At the time this work was in progress, Mr. C. P. Dunn, M-ASCE, was President of International Engineering Company, Inc.; Mr. D. J. Bleifuss, M-ASCE, was Chief Engineer; and Mr. Torald Mundal, M-ASCE, was Chief Design Engineer. Engineering operations in the field were performed under the direction of Mr. W. A. Hohlweg, M-ASCE, and the Late Mr. Gilbert Waddell. Consulting Geologist was Mr. Frank Nickell. Engineering Consultants, who advised on both design and construction and furnished an invaluable service, were the late Mr. James B. Hayes, M-ASCE, and Messrs. John L. Savage, M-ASCE, James P. Growden, M-ASCE, and John S. Cotton, M-ASCE.

DISCUSSION

F. L. LAWTON,[2] F. ASCE.—Kajakai Dam is a classic example of one of the outstanding characteristics of the rockfill dam: its applicability to locations remote from sources of supply for equipment, cement and lumber but with ample rock and sufficient volumes of soil suitable for an impervious core. Kajakai is also typical of another characteristic: broad flexibility of design permitting most economic combination of available indigenous materials.

The overall cost of the dam, including all structures and appurtenant work constructed in the first stage, is given as about three dollars per cubic yard for a total quantity of earth and rock in the main embankment of 4,200,000 cubic yards. This would seem to be an extremely low cost considering the relative isolation of the site, but is no doubt explained by the composition of the work force, with only 69 foreign personnel, presumably all supervisory staff, and 1850 native workmen. It would add to the value of the paper if the author would indicate the relative productivity of the Afghan labour as compared with, say, average American labour of the same categories, and the relative rates of remuneration.

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[2] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

G. F. SUDMAN,[3] F. ASCE.—Mr. Lawton mentions the desirability of comparing the relative productivity versus remuneration of Afghan labor as compared to American labor. Unfortunately the records which might produce any factual information are not available. Many factors would enter into such a comparison such as costs of food and housing, costs of training native personnel, and the somewhat inefficient use of cheap labor that is often trained to perform a single function. It was found, however, that properly trained Afghan personnel, paid only a fraction of what an American would receive, often proved to be practically as efficient as Americans in performing the particular operations for which they were qualified.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3074

ROCKFILL DAMS: LEMOLO NO. 1 DAM

By J. C. Boyle[1] and W. R. Barrows,[2] Fellows, ASCE

SYNOPSIS

This paper describes the Lemolo No. 1 Rockfill Dam of the California Oregon Power Company. The dam, which was built during the winter of 1953 - 1954 and the spring of 1954, is a deck type with a reinforced concrete deck. The reservoir was partially filled in 1954 and was filled in 1955. The dam is relatively small and is of interest primarily because the north abutment was founded on soil consisting of a mixture of old talus, ancient streambed material and clay. A deck type rockfill dam founded on soil is unusual, if not unique.

General Description of Development

The Lemolo No. 1 Dam is the storage and diversion dam for the Lemolo No. 1 hydro-electric development of the California Oregon Power Company. The dam is located on the North Umpqua River about 25 miles northwest of Crater Lake at an elevation of 4000 feet above sea level. Its purpose is to divert water into the opwn power conduit leading to the power plant and to provide 12,000 acre-ft of seasonal storage of the Lemolo No. 1 development and for four downstream developments. The water regulated by this dam falls through a net effective head of 2140 feet in the five plants served by it. The North Umpqua River has a drainage area of 175 miles and an average flow of 394 cfs at the damsite.

The dam is a deck type rockfill having a reinforced concrete deck. (See Fig. 1) It has a maximum height of 120 feet and a length of 775 feet along the crest. Because of the soil foundation on the north abutment, unusual measures were taken to protect the north abutment from erosion and extra

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Note.—Published essentially as printed here, in December, 1959, in the Journal of the Power Division, as Proceedings Paper 2285. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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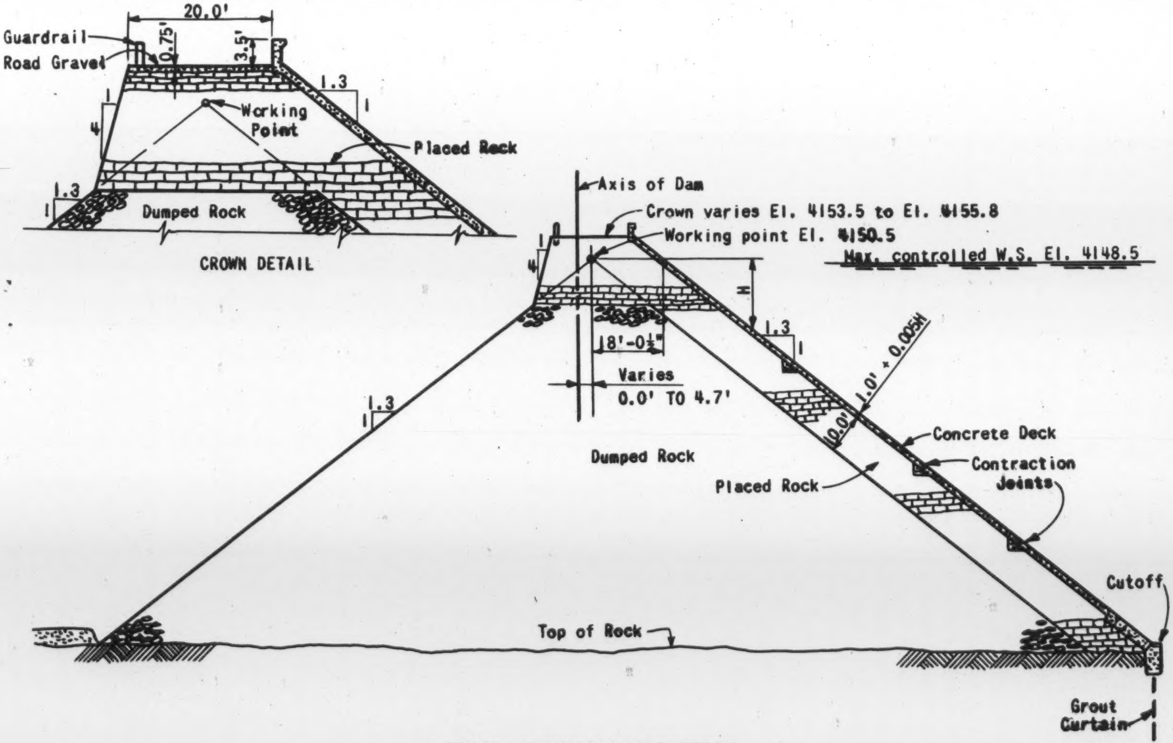
20.0'
Guardrail
Road Gravel
0.75'
3.5'
1.3
1
1
4
Working Point
Placed Rock
1.3
1
Dumped Rock
CROWN DETAIL
Axis of Dam
Crown varies El. 4153.5 to El. 4155.8
Working point El. 4150.5
Max. controlled W.S. El. 4148.5
1
4
H
1.3
1
18'-0½"
Varies
0.0' TO 4.7'
1.0' + 0.005H
10.0'
1.3
1
Dumped Rock
Concrete Deck
Contraction Joints
Placed Rock
Cutoff
Top of Rock
Grout Curtain

FIG. 1.—TYPICAL DAM SECTION

precautions were taken in the construction of the concrete deck to prevent cracking and the accompanying leakage. Excessive leakage around the cutoff or through the deck, or heavy precipitation flowing down through the loose rockfill could become concentrated and erode the soil foundation of the north abutment and cause failure or severe damage, if not prevented or collected and safely carried out of the fill.

A rockfill was selected because the foundation was not considered suitable for a concrete dam and material for compacted fill could not be located within economical hauling distance. The deck type was selected because material for an impervious compacted membrane could not be located within economical hauling distance. As is usual, economic considerations played an important role in the selection of the type of dam. In this case, however, local conditions made it unnecessary to prepare comparative estimates to select the most economical type which could be depended upon to safely perform its function.

Outlets

Two outlets for the controlled release of water were provided. A low level sluice capable of releasing water for downriver developments when the power plant is shut down, was located at streambed level. The power outlet designed to control flow to the open power conduit was located on the north abutment at the highest level which would permit releases when the storage reservoir was drawn to its lowest operating level.

The low level sluice conduit is a 9 feet diameter cast in place reinforced concrete pipe through the fill which was used for stream diversion during construction. After diversion was completed, a 60 inch Howell-Bunger valve was installed at the lower end of the conduit. The Howell-Bunger valve was protected by a 66 inch butterfly valve. A 12 inch Howell-Bunger valve was installed to return water to the streambed for fish. The intake is equipped with trashracks and a slide type gate which can be used to unwater the conduit through the dam.

The power outlet conduit is a 7 feet 6 inches diameter cast in place reinforced concrete pipe through the fill. Flow is controlled at the downstream end by two 60 inch Howell-Bunger valves. The intake is equipped with trashracks and a wheeled gate capable of closing against full flow through the conduit.

Spillway

The spillway was excavated through the rock on the south abutment and consists of a channel about 800 feet long with a control section located on the axis of the dam. The control section was lined with concrete, but the remainder of the rock channel was not lined. The control section consists of three 33 feet 4 inch long bays separated by piers. The two outboard bays are equipped with 3 feet high flashboards and the center bay is equipped with a 10 feet high tainter gate. The spillway crests are located to provide a discharge capacity of 16000 cfs with 3 feet of freeboard on the dam. Opening the low level sluice and the power outlet will increase the discharge capacity to 18000 cfs. Rock from the spillway excavation meeting the requirements for fill material was used in the fill.

Foundation Explorations

The foundation was explored by making a few core drillings at strategic points and digging a number of surface exploration trenches. Except for the upper part of the north abutment, the rock was exposed or covered with only a few feet of overburden. Anyone familiar with the extremely irregular formations in this volcanic area will appreciate that a great many core borings are unwarranted as the information produced soon reaches the point of diminishing returns.

Geology

All the rocks in the region have been formed directly or indirectly by volcanic action. The formations are characterized by their extreme irregularity produced by the effects of intermittent lava flows, stream erosion and deposition, and ash deposits.

As exposed by the exploration program, the foundation stripping and the cutoff excavation, the foundation at the streambed and part way up the abutments consists of alternate layers of dense hard basalt and basaltic agglomerate. (See Fig. 2). On the south abutment beginning at about 60 feet above streambed there is an 18 feet layer of well cemented laminated impervious tuff. Above the tuff the alternate layers of agglomerate and basalt continue well above the top of the dam.

On the north abutment beginning about 40 feet above streambed there is a 16 feet layer laminated tuff of low permeability but which is not as well cemented as that on the south side.

Above the tuff there is only soil consisting of a mixture of old talus, ancient streambed material, and basalt boulders mixed with clay and having a low permeability. The photograph Fig. 3 shows the north abutment after the stripping was completed and excavation of the cutoff started. The dark nearly horizontal strata in the center of the picture is the tuff. The lighter material above the tuff is soil. A few boulders embedded in the soil are visible.

Rock Fill

The specifications for the rock to be used in the dam required that it should be hard and durable and that it should be free from cracks or incipient jointing which would cause it to shatter when dumped in the fill. They also required that it should resist crushing under load and be of such a mineral composition as to resist disintegration under the action of the elements. The specifications further required that the grading be such as to produce a dense fill with rock sizes ranging from 1 to 10 tons predominating in the fill. It was further required that not more than five percent of the fill should consist of fines less than 4 inches in size.

The rock used in the fill is all basalt and was obtained from a quarry on the south abutment about 1/2 mile downstream from the dam, and from the spillway excavation. The fill is of conventional design and was built in two zones consisting of a dumped fill making up the main body of the dam, and a more dense placed fill upon which the concrete deck rests.

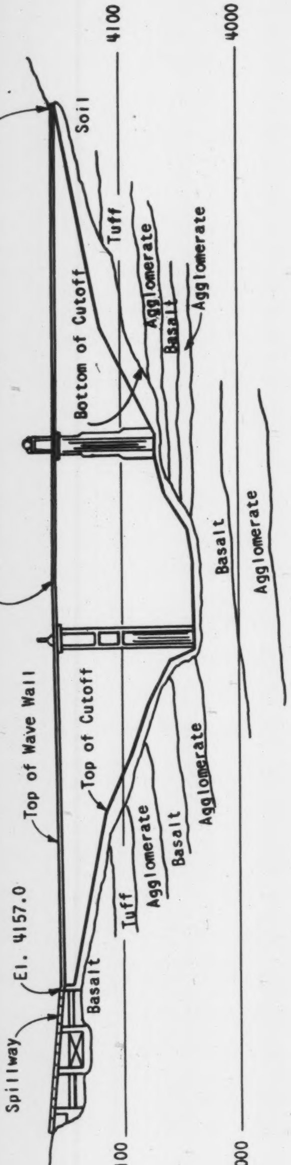
4100
4000
El. 4157.0
Soil
Tuff
Agglomerate
Agglomerate
Basalt
Agglomerate
Bottom of Cutoff
El. 4159.3
Basalt
Agglomerate
Top of Wave Wall
Top of Cutoff
Agglomerate
Basalt
Agglomerate
Agglomerate
Tuff
El. 4157.0
Basalt
Spillway
4100
4000

FIG. 2.—GEOLOGIC SECTION ALONG CUTOFF

Fig. 4 is a photograph of the upstream face of the dam during construction of the placed rock fill and drilling for the grout curtain. Construction of the power outlet control tower is also in progress. Fig. 5 is a photograph of the upstream face of the rock fill in a more advanced stage. In the left foreground can be seen the sluice outlet tower and the timber crib division structure.

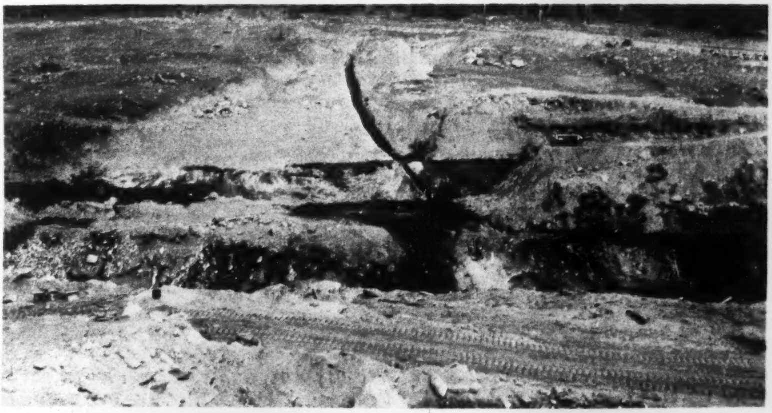

FIG. 3.—NORTH ABUTMENT

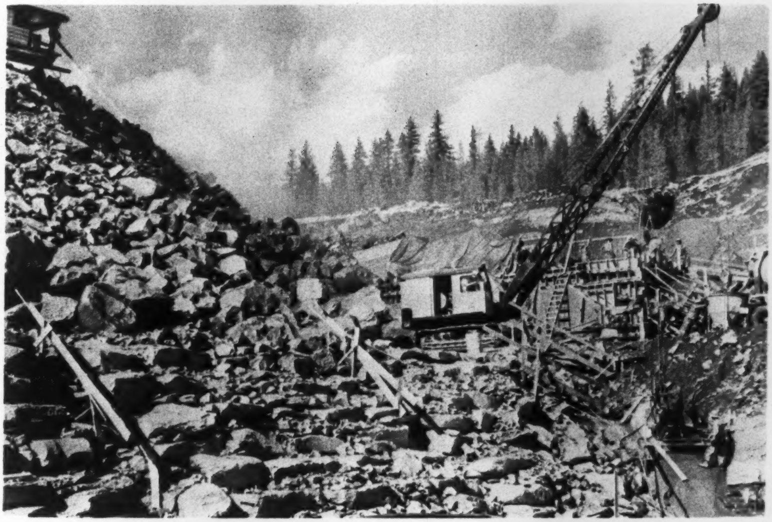

FIG. 4.—PLACING ROCKFILL

The dumped fill was placed in three lifts. The conduits through the fill were protected by covering them with a minimum of 30 inches of quarry fines covered with 4 feet of placed rock before starting dumping operations. All dumped fill was sluiced as it was dumped, using three volumes of water to each volume of rock.

The 10 feet thick placed rock fill was constructed of selected large stones carefully placed and chinked to obtain the maximum contact between placed stones and between the placed fill and the dumped fill. After the placed fill was completed to grade the surface voids were filled with gravel to the maximum extent possible in order to minimize loss of cement when placing the deck.

In the area where the fill was placed in rock foundation, most of the loose overburden material and all the organic matter was removed. On the north abutment where the fill was placed on soil, the surface material was stripped to an average depth of ten feet to remove all stumps, roots and loosely compacted surface materials.

The dam was built with a slight camber upstream and vertically. The canyon was nearly "V" shaped and both the horizontal and the vertical cambers were laid out with straight lines intersecting at a point at the center of the dam. The horizontal camber was 4.7 feet and the vertical camber was 2.3 feet.

Cut-Off Wall

When in contact with rock, the cut-off wall has a minimum thickness of 3 feet and was keyed into the rock a minimum of 5 feet. The wall was permitted

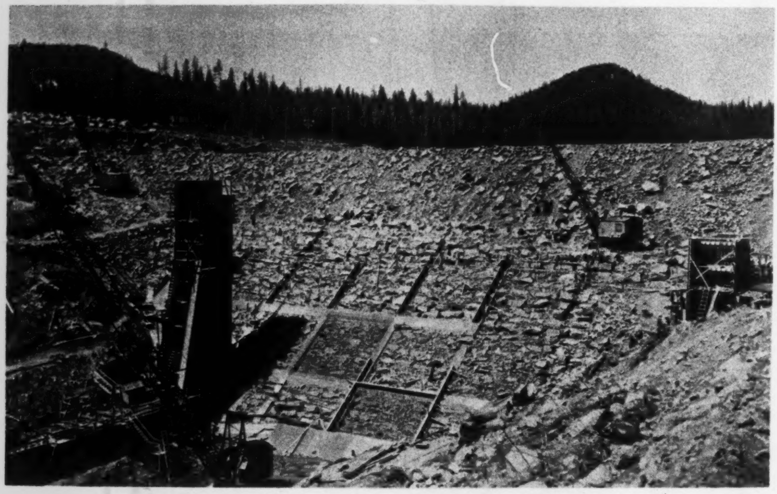

FIG. 5.—ROCKFILL

to project 3 feet above the rock surface without special treatment. For projections exceeding 3 feet, the voids in the placed rock were filled with sand cement grout to form a lean rubble mass in contact with the cut-off wall.

On the north abutment where the wall did not extent to rock, its thickness was increased to a minimum of 4 feet. Three criteria were established to determine the bottom of cut-off.

1. When terminated in the tuff, the minimum depth of cut-off below the stripped foundation was 0.375 times the head on the foundation surface.
2. When terminated above the tuff the minimum depth of cut-off was equal to the head on the stripped foundation surface.
3. When the cut-off was extended into the foundation a distance less than the head on the stripped foundation, it was required to penetrate a minimum of 5 feet into the tuff.

These requirements established a definite minimum line for the bottom of the cut-off and provided a vertical creep distance of 0.75 times the head or a weighted creep ratio of 2.25 in the tuff. In the soil the vertical creep distance was twice the head, giving a weighted creep ratio of 6. As a further precaution against piping, the soil foundation was covered with an inverted filter blanket.

Grout holes were generally drilled and grouted from the bottom of the cut-off trench. The group curtain was extended down a distance equal to the head on the top of the hole. Spacing of holes was determined by field conditions and the grout take of adjacent holes. A total of 78 holes were grouted giving an average hole spacing of 11 feet. The average hole took 34 sacks of cement and individual holes varied from 6 to 302 sacks of cement. A number of holes were stage grouted to minimize surface leakage.

Concrete Deck

The concrete deck, details of which are shown on Figs. 6 and 7 was 12 inches thick at the top and increased in thickness 0.005 feet for each foot of

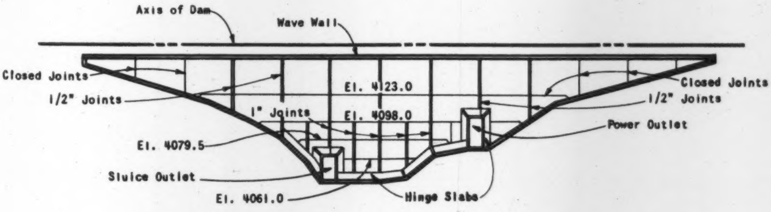

FIG. 6.—PLAN OF CONCRETE DECK

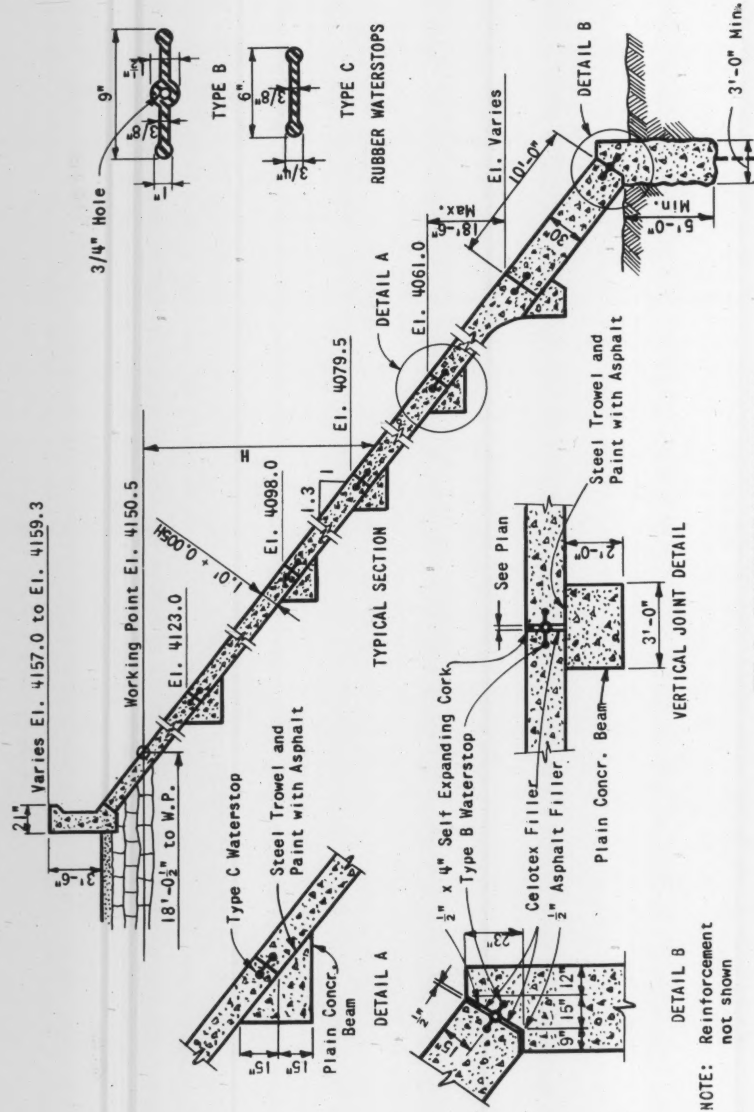
3/4" Hole
9"
1/2"
3/8"
1"
TYPE B
6"
3/8"
3/4"
TYPE C
RUBBER WATERSTOPS
DETAIL B
3'-0" Min.
5'-0" Min.
El. Varies
10'-0"
18'-6" Max.
30
DETAIL A
El. 4061.0
El. 4079.5
H
Steel Trowel and Paint with Asphalt
1
1.3
El. 4098.0
1.0' + 0.005H
Varies El. 4157.0 to El. 4159.3
Working Point El. 4150.5
El. 4123.0
TYPICAL SECTION
21"
3'-6"
18'-0½" to W.P.
Type C Waterstop
Steel Trowel and Paint with Asphalt
Plain Concr. Beam
15"
15"
DETAIL A
See Plan
2'-0"
3'-0"
Steel Trowel and Paint with Asphalt
½" x 4" Self Expanding Cork
Type B Waterstop
Celotex Filler
½" Asphalt Filler
Plain Concr. Beam
VERTICAL JOINT DETAIL
½"
23"
15"
12"
15"
9"
DETAIL B
NOTE: Reinforcement not shown

FIG. 7.—CONCRETE DECK DETAILS

height. This resulted in a deck thickness of about 18 inches at the bottom.

The deck was reinforced with 0.50 percent reinforcement in both directions in the center of the slab. In the lower portions of the dam a 10 feet hinge slab spanning from the cut-off wall to the rock fill was provided. Hinge slabs were also used around the conduit intakes, the slabs spanning from seats on the intakes to the rock fill. The hinge slabs were 30 inches thick and reinforced in both faces. Designed structural reinforcing was located in the bottom of the slab and temperature reinforcing in the top.

The concrete deck was divided into small sections by contraction joints for ease of placing and to control cracking. Below elevation 4098 the contraction joints formed the deck into squares 30 feet in a side or into smaller areas as required by the topography. Above elevation 4098 the contraction joints divided the deck into rectangles 60 feet long and varying in height from 40 to 50 feet.

All contraction joints were sealed with rubber water stops. Three basic types of joints were provided. The joints between the deck and cutoff wall and between the deck and outlet structures were filled with 1/2 inch joint filler and had water stops capable of withstanding large movements. The vertical contraction joints had joint filler varying from one inch at the center of the dam to zero at the abutments and water stops capable of taking large movements. The horizontal joints were cast without joint filler and had water stops capable of taking small movements.

The photograph labeled Fig. 8 was taken during the placing of one of the deck slabs. Fig. 9 shows the completed concrete deck just prior to closure of the diversion.

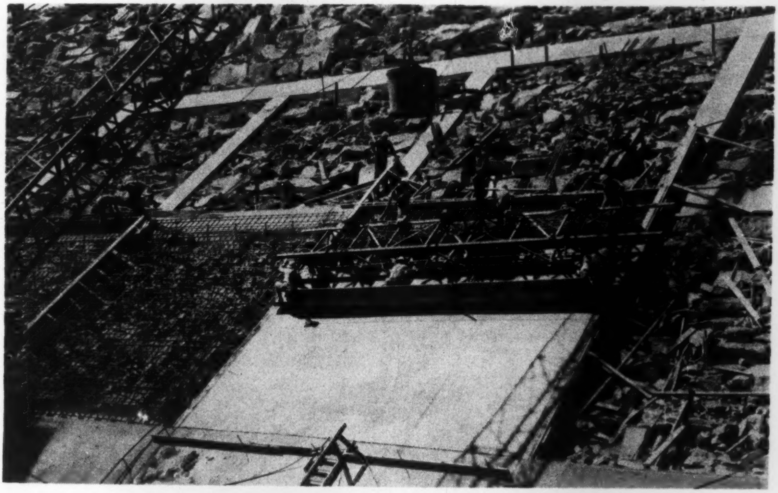

FIG. 8.—PLACING CONCRETE DECK

Drainage Provisions

A drainage system was designed to protect the soil on the north abutment from erosion which could be caused by excessive seepage around the cut-off, excessive leakage through the deck or from heavy precipitation running down through the fill. Water from any of these sources, if concentrated into rivulets running down the slope to the riverbed could erode the foundation and cause serious damage to the dam. The system was also useful in collecting sluicing water and transporting it out of the fill during rock placing operations.

The drainage system was divided into two parts. An inverted filter blanket was located under the placed rockfill and extended from the power outlet structure to the top of the dam. It was intended to intercept seepage and prevent piping around the cut-off. In addition, a series of french drains were constructed normal to the axis of the dam. They were designed to intercept and collect any water flowing on the surface of the foundation. Perforated pipe from the french drains was extended into the coarse gravel of the filter blanket and a collector drain at the toe of the rock fill carried all the water to a point where flow from the drainage system could be observed.

The inverted filter consisted of three 8 inch layers of graded material. The coarse sand placed in contact with the undisturbed foundation had a critical size of 1.0 mm. The fine gravel which followed had a critical size of 1/4 inch. The top layer consisted of coarse gravel having a critical size of 1-1/2 inches. During placing of the dumped rock fill the filter blanket was protected by a layer of placed rock having the voids filled with sand cement grout.

The french drains consist of perforated tile laid in trenches spaced at about 20 feet centers. The trenches were backfilled with coarse gravel which

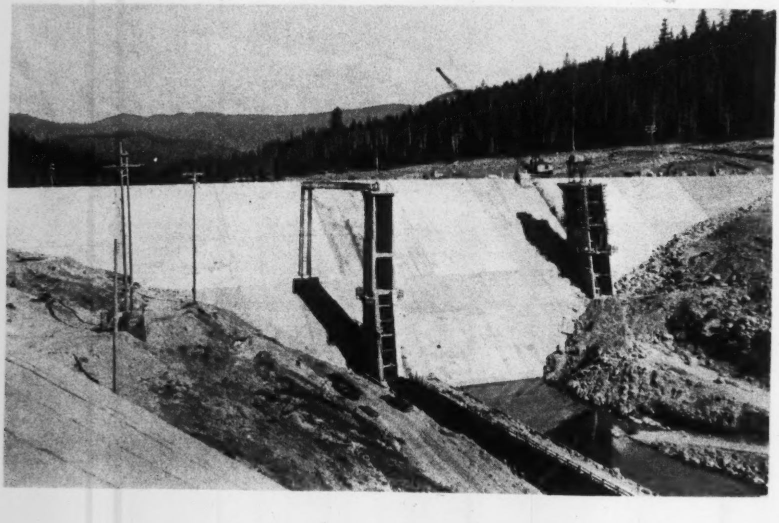

FIG. 9.—COMPLETED CONCRETE DECK

completely surrounds the perforated pipe. The trenches were filled with gravel to the ground surface and the rockfill placed directly on the gravel, thus insuring that any surface water can readily find its way to the drain tile.

The power conduit was located at the top of the hard rock and formed a barrier to the flow of water down the slope to the riverbed. To drain this pocket, the uphill slope of the trench excavated for the conduit was blanketed with gravel and a perforated tile drain was located at the bottom of the trench on the uphill side of the conduit.

Settlement

To provide permanent points for the measurement of displacements of the concrete deck, brass plugs were located at the four corners of each slab. Periodic measurements of the movements of these plugs were made between July 1, 1954, and May 6, 1955, covering the first cycle of reservoir filling and emptying. The maximum horizontal displacement of 0.245 feet was measured at a point about 35 feet below the crest of the dam. Displacements along a line 60 feet below the crest were of almost the same magnitude. The maximum settlement of 0.359 feet was measured at a point 60 feet below the crest. Settlements along a line 35 feet below the crest were of slightly smaller magnitude. Displacements above and below the two lines mentioned were smaller.

On Fig. 10 is plotted some of the results of the measurements which have been as consistent as could be expected. A typical settlement curve of a point on the high part of the fill is plotted against time for comparison with the reservoir level. Also included are the horizontal and vertical displacements along horizontal lines across the deepest part of the valley. These measurements were made on May 6, 1955, just before the second reservoir filling cycle started. Since that time, a wet cycle has kept the reservoir at a higher level than normal and no measurements have been made. It is believed that displacements since May 6, 1955, have been small.

Cracking of Concrete Deck

No cracks have been found in the concrete deck. The reservoir has not been lowered below elevation 4080 since the first filling so the lower part of the slab has not been examined. It is estimated from observation of the leakage through the dam that deck cracking, if any, is very minor in character.

Leakage

There has been a very limited amount of leakage through the north abutment which has been collected in the drainage system. There has been no leakage through the main dam structure. Such leakage between the two outlet conduits would show up in the stream bed at the toe of the dam.

There is a leak of about 1 cfs through an open seam in the rock on the south abutment. This leak appears in the surface near the lower end of the spillway channel excavation when the reservoir is full, and disappears when the reservoir is lowered 14 feet. The reservoir entrance to this open seam has not yet been located and it will be closed when uncovered.

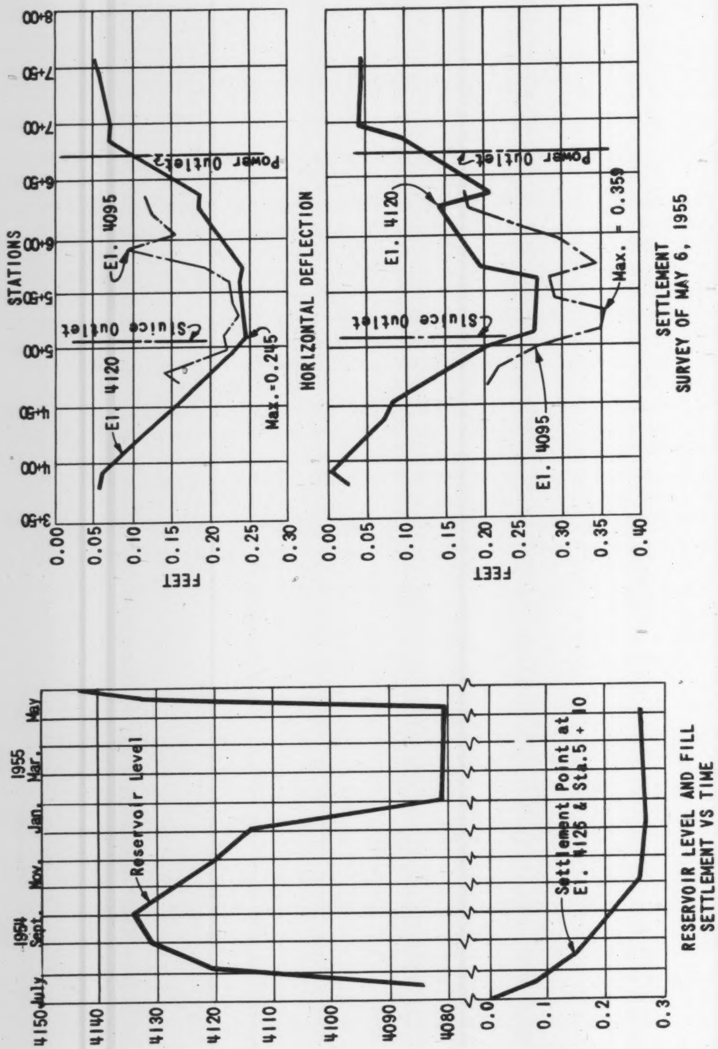

FIG. 10.—MOVEMENT OF FILL

There have been no unexplained increases or decreases in the leakage through the dam and it is believed that the cutoff and drainage provisions combined with the crack prevention program for the deck has produced satisfactory results.

ACKNOWLEDGMENTS

The Lemolo No. 1 Dam was designed by Pioneer Service and Engineering Co., under the direction of Mr. Stephen Wehner, who was then Chief Hydraulic Engineer. Construction was by the Morrison-Knudsen Construction Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 3075

ROCKFILL DAMS:
CHERRY VALLEY CENTRAL CORE DAM

By H. E. Lloyd,[1] F. ASCE, O. L. Moore,[2] M. ASCE, and
W. F. Getts,[3] F. ASCE

With Discussion by Messrs. Joseph C. Dodd; F. L. Lawton; and
H. E. Lloyd, O. L. Moore, and W. F. Getts

SYNOPSIS

This paper describes the construction of Cherry Valley Dam of the Hetch Hetchy Water Supply of the City and County of San Francisco. Cherry Valley Dam is of the central core rockfill type. The central core is composed of compacted decomposed granite and the rockfill is of freshly quarried granite, placed in lifts of 15 to 30 feet and sluiced. The performance of the embankment during the first two fillings of the reservoir is also described and measurements of the settlements and deflections are given.

EMBANKMENT DESIGN

The design of the Cherry River Project of the City and County of San Francisco has been presented in detail in Convention Preprint Paper No. 71, "The Cherry River Project of the City of San Francisco" by M. L. Dickinson,

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Note.—Published essentially as printed here in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1733. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[2] Constr. Engr., Hetch Hetchy Water Supply, Power and Utilities Eng. Bureau, City Public Utilities Comm., San Francisco, Calif.

[3] Senior Civ. Engr., Public Utilities Comm., City and County of San Francisco, Calif.

M. ASCE. (March, 1953). For the benefit of those who do not have access to this paper the design of the dam embankment will be reviewed briefly.

Cherry Valley Dam is of the composite earth and rock-fill type, with the impervious core centrally located in the cross-section. A two layer transition zone, 20 feet in total thickness, is placed between the impervious core and the rock envelope, both upstream and downstream. The dimensions are as follows:

Crest length, feet	2,600
Crest elevation, feet	4,715
Crest height above streambed, feet	315
Depth streambed to bedrock, feet	15
Height, total, feet	330
Crest width, feet	40
Thickness at base, maximum, feet	1,320
Volume, cubic yards	
impervious core	2,932,000
transition zones	593,000
rock envelopes	3,475,000
Total	7,000,000

This type of dam was selected in order to make the best possible use of local materials. Ample deposits of material for the impervious core were found within two miles of the damsite, and excellent granite for the rock envelopes was obtained from a quarry on the right abutment, just downstream of the dam. Material for the transition zones was obtained from deposits in the reservoir area. The embankment section was proportioned to use as much core material as possible while still maintaining the desired factors of safety under the various conditions of loading. A core with the required degree of impermeability could have been obtained with a thinner section, but since the cost per cubic yard of the rock fill was about twice that of the core, it was desirable to use a section containing the maximum proportion of core. However, if the embankment were to be built entirely of the core material, the slopes of the upstream and downstream faces of the embankment would have to be flattened to maintain the required stability of the section. This would increase the total volume of material required for the embankment and also the length of the combined outlet and diversion tunnel. In the case of Cherry Valley Dam the total cost of the embankment would remain about the same since the increase in volume would be offset by the lower unit cost of the material, but the cost of the tunnels would be increased about 50% due to the greater length required.

The outside slopes of the rock fill were made as steep as possible while maintaining the required stability. The overall slope of the outside face of the rock is 2 horizontal to 1 vertical. The upstream slope of the impervious

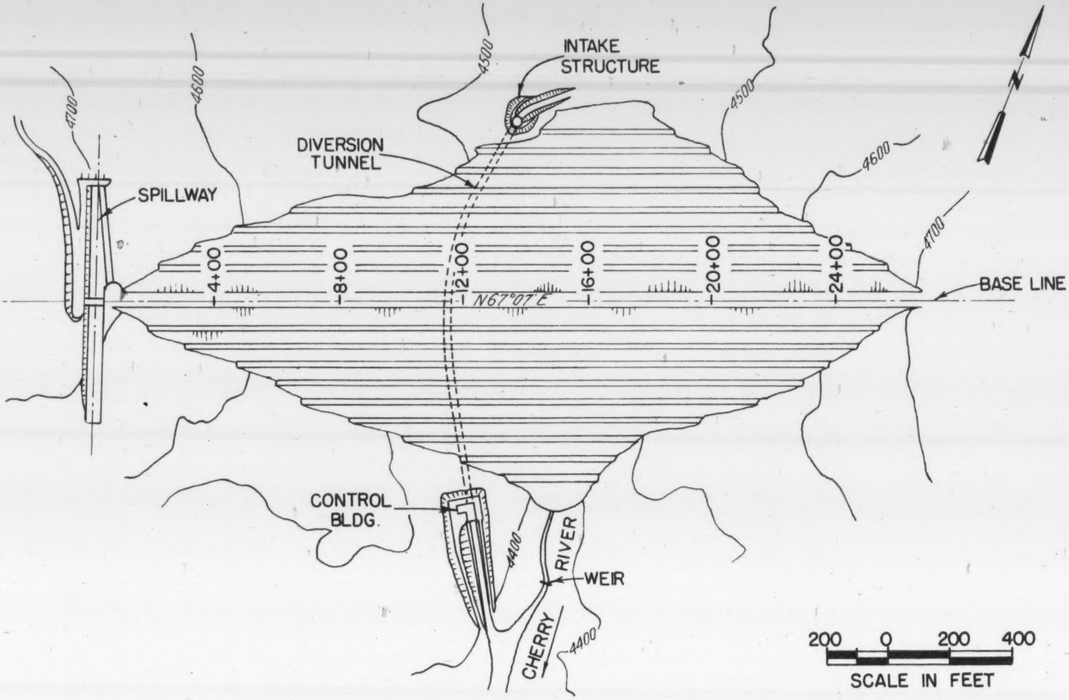

INTAKE
STRUCTURE
DIVERSION
TUNNEL
SPILLWAY
4700
4600
4500
4500
4600
4700
BASE LINE
4+00
8+00
12+00
16+00
20+00
24+00
N67°07'E
CONTROL
BLDG.
4400
RIVER
WEIR
CHERRY
4400
200 0 200 400
SCALE IN FEET

FIG. 1.—CHERRY VALLEY DAM, GENERAL ARRANGEMENT

core is 0.70 to 1 while the downstream face is 0.75 to 1. The difference of 0.05 in the upstream and downstream slopes represents about 160,000 cubic yards of material. If this material were to be rock instead of decomposed granite it would cost about $1.00 per cubic yard more, so that the difference in the core slopes represents about $160,000.

The borrow areas for the impervious material were explored by means of auger holes at 250 foot centers. Samples were taken at intervals of not over seven feet in these holes, or at changes in type of material. Sieve analyses and moisture density tests were made on these samples primarily to determine the types of material in the borrow area and the uniformity of these types. In the selected borrow area it was found that the material could be classified into four groups or types. All of the material consisted of decomposed granite classified as a silty sand but variations in the proportions of feldspar and quartz in the original granite caused variations in the residual material.

Composite samples of each of the four types of material were tested for gradation, plasticity, specific gravity, compaction characteristics, direct shear strength, permeability, and consolidation characteristics. From the results of these tests values were assumed for the design of the embankment, as follows:

dry weight	106 lbs. per cu. ft.
saturated weight	131 " " " "
angle of friction	27 degrees
cohesion	0

These values represent the minimum quality of the material available for the core. The average quality is somewhat better. This represents an additional safety factor which was not taken advantage of in the design.

The rock was not tested other than by core drilling of the prospective quarry areas. Based upon tests reported by others, the following design values were used for the rock fill:

dry weight	110 lbs. per cu. ft.
saturated weight	132 " " " "
angle of friction	45 degrees

The local stability of the rock slopes between berms was checked by the infinite slope method. These slopes had factors of safety ranging from 1.33 with empty reservoir to 1.10 with full reservoir and 0.05 g earthquake forces acting.

The overall embankment was checked by use of the slip circle method. The minimum factors of safety for the various design conditions are:

Normal operating conditions	1.50
Infrequent operatiing conditions such as sudden drawdown	1.25
Normal operating conditions plus 0.05 g earthquake	1.10

The transition or filter zone between the core and the rock consists of two layers or zones. The finer layer, adjacent to the core, is 8 feet thick while the coraser layer is 12 feet thick. The gradation of the material in these zones was based on the criteria that:

1. To prevent migration of the finer material into the coarser material the 15% size of the coarser material should not be greater than 5 times the 85% size of the finer material;

and

2. For drainage the 15% size of the coraser material should not be less than 5 times the 15% size of the finer material.

As an added protection against migration of the fine material into the rockfill, selected fine material from the quarry was placed adjacent to the transition zone. This material in effect acts as a third transition zone and actually reduced the cost of the embankment since the material would otherwise have been wasted.

During construction the river was diverted through a 16 foot diameter concrete lined tunnel under the right abutment. Upon completion of the embankment an intake tower was constructed at the upstream end of the tunnel and outlet valves were placed at the downstream end. A steel lining, 15'-4" inside diameter, was placed in the tunnel from the line of the grout curtain to the downstream end. The space between the steel lining and the concrete was filled with a sanded grout. A tee and a butterfly valve were also installed at the downstream end to serve the future power development.

A side-channel type spillway is provided on the right abutment. Model studies were made to determine the most efficient layout of the spillway and discharge channel. The crest of the concrete ogee weir is at elevation 4700, 15 feet below the nominal crest of the dam. The spillway will pass a flow of 52,000 cfs with 3 feet of freeboard on the dam.

Construction

Foundation Preparation

In October of 1951 a contract was awarded to the Owl Truck and Construction Company in the amount of $684,480.00 for clearing and stripping of dam site foundation, cutoff trench, excavation and cutoff grouting. The entire foundation area was stripped to weathered bedrock, final clean up being performed by hydraulic monitors. A total of 291,500 cubic yards of overburden was removed. A cutoff trench to relatively firm, unweathered granite was excavated in the foundation adjacent to the dam axis. The trench averaged 20 feet in width at the bottom and 11 feet in depth, varying from 20 feet deep at the abutments to 4 feet deep at the stream bed. Location of the cutoff trench with respect to the dam axis is shown in Fig. 2. After completion of cutoff trench excavation, a single line of cutoff grout holes on five foot centers was drilled and grouting by stages down to 120 feet in depth. Primary holes were drilled and grouted on 20 foot centers, then secondary holes were drilled and grouted between primary holes, reducing the hole spacing to 10 feet. Finally, tertiary holes were drilled between primary and secondary holes, reducing hole spacing to 5 feet. In general, grouting was performed on all holes so that stages in adjacent holes differed by no more than one stage. In all cases

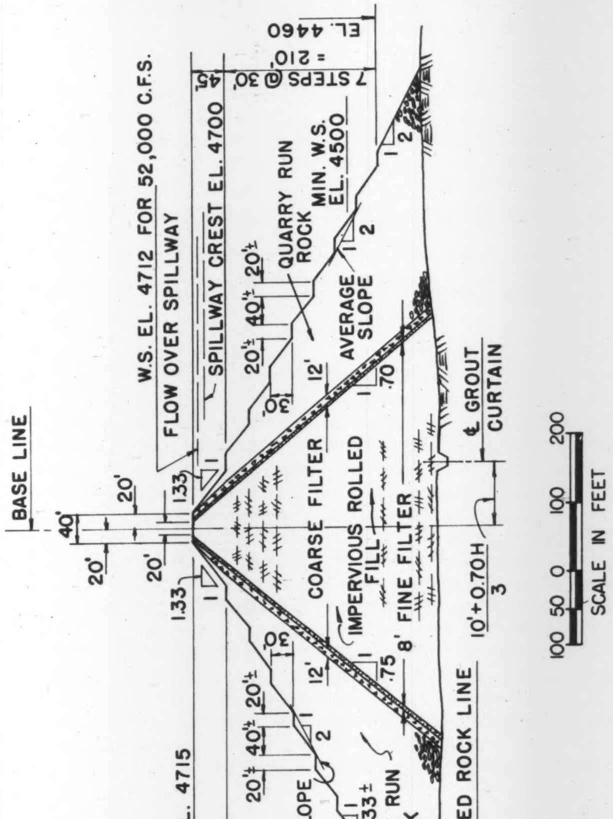

BASE LINE
40'
20'
20'
20'
20'
W.S. EL. 4712 FOR 52,000 C.F.S.
FLOW OVER SPILLWAY
SPILLWAY CREST EL. 4700
45'
7 STEPS @ 30' = 210'
EL. 4460
DAM CREST EL. 4715
8 STEPS @ 30' = 240'
45'
EL. 4430
1.33
1
1.33
1
20'±
40'±
20'±
30'
12'
QUARRY RUN ROCK
MIN. W.S. EL. 4500
AVERAGE SLOPE
1
2
AVERAGE SLOPE
1
.70
1
2
COARSE FILTER
IMPERVIOUS ROLLED FILL
FINE FILTER
8'
1
.75
AVERAGE SLOPE
1
1.33±
QUARRY RUN ROCK
1
2
WEATHERED ROCK LINE
10'+0.70H / 3
℄ GROUT CURTAIN
100 50 0 100 200
SCALE IN FEET

FIG. 2.—TYPICAL SECTION

grout used was neat cement grout with water-cement ratios ranging between 0.6 to 1 and 10 to 1. Grout pressures used averaged one pound per square inch per foot depth of hole. Grout takes (in cubic feet per linear foot) in the various stages are shown in following table:

Stage Hole	First 0 to 23'	Second 23' to 43'	Third 43' to 63'	Fourth 63 to 123'
Primary	2.366	0.745	0.097	0.056
Secondary	1.068	0.332	0.000	—
Tertiary	0.590	—	—	—

Additional foundation preparation under this contract consisted of backfill of prominent seams or crevices with a lean concrete under the compacted earth core to facilitate fill placement.

Dam Embankment

In October 1953, a contract for construction of the dam embankment in the amount of $7,162,800 was awarded to the Guy F. Atkinson Company. As soon as the necessary equipment could be assembled, a small quarry was developed just upstream from the damsite and construction of an upstream rock cofferdam commenced. This cofferdam was designed and constructed to be later incorporated into the upstream toe of the rock embankment. Final closure and diversion of the Cherry River was accomplished in June 1954, with the placing of a 30 foot thick blanket of impervious material on the upstream face of the cofferdam. The first lift of this impervious material was placed to an elevation about 30 feet above streambed by end dumping and dozing into comparatively still water. The impervious blanket was completed to the crest of the cofferdam, 70 feet above streambed, by placing in lifts about 2 feet thick. Compaction of the impervious blanket was limited to that obtained by hauling and spreading equipment. Seepage through the completed cofferdam was minor and was handled by a 1000 gpm electric pump.

Due to the slope of the streambed at and downstream from the damsite, as well as the low flow diverted (a maximum of 3200 cfs during construction in the first season), no downstream cofferdam was required.

Compacted Earth Fill

Under the terms of the contract, the Contractor was allowed the option of utilizing either one or both of two designated impervious borrow pits, either of which contained sufficient material for the embankment, as determined by auger borings. Both borrow pits offered a downhill haul to the damsite, and the Contractor chose to haul all of the fill material from a single pit, distant about three miles from the damsite, only slightly less than the distance to the alternate pit. The selection of pits provided a saving in haul road construction, inasmuch as the haul road constructed passed just below the major rock quarry, one-half mile from the Dam, and was used also for trucks from the quarry. This arrangement proved satisfactory, although it was necessary to provide traffic control at the intersection of the quarry road and the main haul road much of the time.

Impervious material in the borrow pit selected by the Contractor was a decomposed granite derived by deep chemical weathering of inplace granitic bedrock. The area was known to contain several springs and was traversed by two small streams which flowed throughout the year. Moisture content of material in place ranged from 5% to 15% over the optimum required for compaction and this single fact to a major extent governed the Contractor's operations both in the borrow pit and on the fill.

Mechanically the borrow pit classification ranged from sandy silt to silty sand with 95% of the material classified as silty sand. Within the above ranges the material was further classified as follows:

Type	Characteristics	% finer than #200 screen	% of total Mtl Used
1.	Finely graded-Low Density	41 to 82	15%
2.	Fine graded-Above average density	34 to 41	35%
3.	Coarsely graded-Below average density	22 to 34	35%
4.	Coarse graded-High Density	10 to 22	15%

The average plasticity index of material in the borrow pit was 2.3 and ranged from 5.0 to 1.0. Of the samples tested 15% of the material had a P.I. of 5.0, 15% a P.I. of 3.0, 35% a P.I. of 2.0 and 35% a P.I. of 1.0.

Laboratory dry densities of the material ranged from a minimum of 113.3 lbs to a maximum of 128.0 lbs. per cubic foot when compacted at modified A.A.S.H.O. effort (56,250 ft. lb. per cu. ft.)

During the winter of 1953-54 the borrow pit, 200 acres in area, was cleared of trees and brush and stripped of humus and organic soil to an average depth of 2 feet. Haul roads were constructed, and some surface drainage excavation was performed.

Under the bidding schedule, the price established by the City for borrow pit excavation was $0.20 per cu. yd. This price included all excavation, whether waste or fill, and covered the cost of treatment to produce material of optimum moisture content, handling of stones and boulders in the pit, loading and hauling. The Contractors' bid price for placing and compacting material in the embankment to the specified density was $0.40 per cubic yard. Compaction results were specified; methods were left up to the Contractor.

In addition to the foundation treatment performed under the contract for stripping the damsite, under the fill contract, all joints and seams under the impervious section of the dam were slush grouted at least 48 hours in advance of placing fill thereon. Thus, foundation slush grouting proceeded up the dam abutments generally two days in advance of fill placement. Slush grout was a sanded grout composed of 1 part cement, 3 parts sand and 2 parts water.

From streambed to the crest of the dam, all impervious material placed against the foundatioh was hand compacted for a distance of from 3 to 4 feet out from the foundation contact, using pneumatic tampers. The Contractor's bid price for this work was $6.00 per hour for each tamper used, maintained and operated, including sufficient hand labor to place material at the contact. A total of 6,430 tamper hours were used during the term of the contract. The area of the contact between the foundation and impervious core was 31 acres. During fill placement, the fill contact zone was kept to an elevation averaging three feet above the balance of the fill.

Impervious material was loaded in the borrow pit, transported to and placed on the embankment by 20 c.y. carryalls operating on the three mile haul road at speeds up to 40 mph. At the maximum rate of placement, about 24,000 c.y. per day, 20 of these units were used on each of two shifts. Carryalls were generally loaded in the borrow pit by two pusher tractors operating in tandem.

Continuous treatment of impervious material in the borrow pit was required to remove residual boulders encountered and to reduce moisture content to optimum for compaction. Material encountered in the pit varied in moisture content from 20% to 30%, and optimum moisture for compaction to specified density varied from 10% to 14%, depending upon gradation and specific density. Reduction of moisture content was accomplished by a systematic cultivation of the borrow pit and rotation on a 4 to 5 day cycle of excavation in designated areas of the pit. The cycle of excavation and treatment was approximately as follows:

> In a designated area where moisture had been reduced to optimum or slightly above, material was excavated to a depth of approximately 18 inches. On the day following, the area was ripped to a depth of about 2 feet by a crawler tractor equipped with a back-mounted ripper, and any boulders encountered were removed to the boundary of the area. On the next day the area was plowed with a two foot bottom plow. This treatment was repeated on the following day in areas of excessively high moisture content. On the next day the area was harrowed to a depth of about one foot by means of a spring tooth harrow. The following day the area was checked for moisture and if found satisfactory, excavated for fill in succeeding shifts.

All treatment for reduction of moisture content was performed on the day shift and the dryer areas of the horrow pit were generally reserved for hauling to the fill after sunset on the swing shift. Varying with the season and time of day, it was determined by testing that moisture content could be reduced by a maximum of 2% during the cycle of rock removal, leveling and sheepsfoot compaction on the fill embankment. Considerable experimentation with compaction units of various sizes and types on lifts of varying thickness was performed in early phases of impervious fill construction. During the early part of the first construction season for impervious fill, Summer, 1954, two rubber-tired compactors were tried, with gross weights of 50 and 100 tons, as well as sheepsfoot compactors ballasted to gross weights varying from 25 to 35 tons. Primarily because of the higher reduction of moisture content due to evaporation resulting from sheepsfoot compaction and secondarily because of the greater apparent compactive efficiency derived from increased weight, the contractor shortly standardized on the 35 ton sheepsfoot roller for all compaction.

A total of 1260 field density tests were taken during construction, averaging one for each 2,300 c.y. of impervious material placed in the embankment. The average field density of all samples taken was 91% of laboratory density attained by the use of apparatus and procedures required for A.A.S.H.O. Method T 99-49, with the following changes:

a) the weight of the compaction hammer was 10 pounds instead of 5.5 pounds.

b) the hammer drop was 18 inches instead of 12 inches

c) the control sample was compacted in 5 equal layers instead of 3, using 25 blows per layer.

To obtain 91% average density the contractor settled on a compaction effort of 12 passes of a 35 ton sheepsfoot roller on each layer of 9 inches, loose depth.

The contractor placed impervious fill in the following sequence:

1. After compaction of a layer, the top surface was bladed off by means of a patrol grader. This operation was found to be essential in obtaining uniform thickness in the subsequent layer of material.

2. Just prior to placement of a layer, the top of the previous layer was scarified to a depth of about two inches to break up hard surfaces caused by the tires of hauling units and to insure bonding of the two layers.

3. A layer of material was placed on the fill by carryalls and spread to a thickness averaging 9 inches, loose measurement, by a crawler tractor equipped with a bulldozer.

4. The loose layer was raked through by a crawler tractor equipped with a rock rake to remove rock of a size over four inches. This operation was supplemented by a continuous patrol of a dump truck and two laborers following behind the rock rake to pick up oversize material which slipped through the rake.

5. The layer was compacted as noted above.

Although the results obtained by the above procedure were specified, the contractor was allowed a great deal of latitude as to methods, and followed the procedure outlined after considerable experimentation.

Transition Zone

The impervious core was flanked up and downstream by a transition zone 20 feet thick grading from sand and fine gravels adjacent to the impervious core to coarse gravel and cobbles. The required gradation for transition material was obtained by selective excavation from extensive deposits of sands and gravels in the flood plain of Cherry Valley. No mechanical grading of the material was necessary.

Transition material was placed in layers one foot thick, each layer being compacted by one complete coverage of a crawler tractor.

During the summer of 1954, placement of impervious and transition fill material progressed to an elevation considerably above the average elevation of the outside rock envelopes.

To retain the transition material on slopes of 0.75:1 and 0.7:1, it was necessary to place rock adjacent to the transition zones as the elevation of the impervious and transition zones increased above that of the full rock section. Rock for this purpose was placed by dumping from the transition zone along the full length of the dam. This "retaining" rock was dumped without sluicing at the point of the dump to prevent damage to transition and core material. Rock dumped dry adjacent to the transition zone reposed on a slope approaching 1: 1. In general, rock selected for dumping adjacent to transition zones was the smallest available from the quarries, and thus furnished additional transition in grading from coarse gravel and cobbles into quarry-run rock.

Rock Embankment

Rock for embankment was granodiorite quarried from three sites, all within a half of a mile from the damsite. Overburden to an average depth of 6 feet was stripped from quarry sites and wasted.

Chemical weathering of the rock extended to an average depth of 35 feet below bedrock surface, the extent and severity of weathering decreasing with depth. During construction of the embankment, some of the more severely weathered bedrock was wasted, although much of it was suitable for rockfill placed adjacent to the transition zones.

For all quarry excavation, including stripping and waste, the Contractor was paid $0.50 per cubic yard, the price being stipulated in the contract by the City, based on cross section measurement of the quarry sites. In addition, for all rock placed in the embankment, the Contractor was paid his bid price of $0.72 per cubic yard, based on embankment cross sections.

The Contractor experimented with various drill hole patterns and loadings and standarized on the following:

1. Drilled 4 inch diameter vertical drill holes, 30 feet deep, on an average spacing of 10 feet by 12 feet. Drills used were wheel-mounted, percussion type, wagon drills.

2. The bottom 4 feet of each drill hole was loaded with 60% gelatin, the balance of the hole with 40% gelatin to within five feet of the collar.

3. Explosives were detonated by means of electric blasting caps, delayed progressively from the working face back into the quarry.

The powder factor used averaged 0.6 pound per cubic yard of rock removed from all quarries.

Blasted rock was excavated from the quarry by means of 5-1/2 cubic yard electric, 3-1/2 and 2-1/2 cubic yard diesel shovels, and very little secondary shooting was required. Fifteen cubic yard, off-highway, rear dump trucks were used to transport blasted rock from quarries to the dam embankment.

In general, the rock fill portion of the dam embankment was constructed by end dumping on 30 foot vertical lifts, although in a few instances 15 foot lifts were used where access became a problem. To obtain the average design rock slope of 2 horizontal to 1 vertical and still save rehandling the surface rock during construction, the up and downstream rock faces were constructed on 20-foot wide horizontal berms at 30-foot vertical intervals, with the rock between berms taking its average material slope of repose of 1.33 horizontal to one vertical (See Fig. 2).

Compaction of rock embankment was obtained by sluicing, using hydraulically operated monitors equipped with nozzles of 3, 3-1/2 or 4 inches diameter, operated at nozzle pressures ranging from 70 to 140 pounds per square inch. The skidmounted monitors were always placed out ahead of the rock dump on top of the previously placed lift 30 feet below, and were situated so that the water stream was directed into the face of the dump.

In dumping, rock was deposited on the edge of the dump and bulldozed over the edge, sufficient fine material being left on the top of each lift to provide an adequate surface for hauling. Prior to placing the next lift, this comparatively smooth surface was ripped up and the fine materials washed down into the embankment. Thus, in addition to sluicing the dump face, primarily at the point of the dump, each monitor was required to sluice the horizontal surface on which the rock was dumped.

Rock embankment was constructed on a two or three shift basis throughout the year, inasmuch as freezing conditions in the winter never became sufficiently severe to halt the work. The maximum rate of rock fill placement attained was about 14,000 cubic yards per day, using ten 15 cubic yard trucks on each of three shifts. Average rate of placement for the construction period was about 7200 cubic yards per working day.

The Dam embankment was completed in October, 1955; total time of construction was two years.

Performance

Reservoir Operation

Since the operation of the reservoir may influence the performance of the embankment, a brief description of the operation to date is given below.

The dam embankment was completed in October 1955. However, the spillway concrete was not completed until the fall of 1956, so that during the 1956 run-off season the water surface in the reservoir was kept below elevation 4650. This elevation is 50 feet below the spillway lip and 65 feet below the nominal crest of the dam. This was possible because the outlet valves have a capacity in excess of normal requirements in order to permit the reservoir to be operated for flood control. This excess capacity allowed the City to maintain the water surface at or below the permissible elevation during the run-off season. Of course, had a flood threatened during this period, it would have been necessary to drown out some of the Contractor's operations temporarily.

In 1957 the reservoir was completely filled. This also allowed a check on the operation of the spillway. Since the maximum inflow at the tailend of the run-off was only around 500 cfs the flow over the spillway was limited to this amount. The maximum elevation of the water surface was 4700.6, 14.4 feet below the dam crest. Under these conditions the spillway functioned without trouble, although the flows and heads really did not offer a very severe test.

In both of these years, 1956 and 1957, it was necessary to draw the reservoir down to elevation 4600 to allow other construction work in connection with the Cherry Power Project to proceed. As soon as the downstream reservoir of the Modesto and Turlock Irrigation Districts could receive the water, the outlet valves were opened. In 1956 the water surface was lowered from elevation 4650 to 4595, a total of 55 feet, in 15 days. In 1957 it was lowered 100 feet in 45 days. These conditions approach the classical case of "sudden drawdown." However, since the water level was kept at its maximum elevation for only a short period of time, the core was not completely saturated, and the drawdown condition was not as severe as it would have been had the core been saturated.

It is expected that the reservoir will fill again this year, and as in the past two years it will again be drawn down to elevation 4600.

Measurements

Before filling of the reservoir was started, survey monuments were set in the impervious core along the crest of the dam. These monuments consist of a standard bronze survey marker set in an 8 inch diameter concrete base, extending 3 feet into the core material. These monuments were set at 400 foot intervals in a straight line along the crest of the dam. Reference points were set on the extension of this line at each abutment. The elevation of each point was determined and recorded. Bench marks were set at each abutment for future reference. The horizontal distances between the reference points on the abutments and the monuments along the dam crest were also measured and recorded. Settlement points were not established on the rockfill, neither on the crest of the dam nor on the berms.

Measurements have been made on these markers at irregular intervals since the reservoir was placed in operation. Readings have been taken rather frequently during run-off period when the reservoir fills quite rapidly and also during the period when it is being drawn at a greater than normal rate. At other times the interval between readings is much longer since conditions are pretty much constant and the embankment movements occur at a much slower rate.

The amount of settlement at each marker is measured. Also the deflection of the marker from a straight line along the theoretical axis of the dam. The horizontal distances from marker to marker have been checked several times to see if there is any apparent movement along the fill from the abutments toward the section of maximum height. To date there does not appear to be any motion in this direction.

A weir was also set in the old stream channel below the dam to measure seepage. This has not been too successful as described in more detail later.

Seepage

Seepage is probably the most direct indication of the success of this type of dam. It has been said that a stable dam embankment can be constructed out of almost any competent material provided that the resultant seepage is tolerable. What the tolerable seepage may be is something to be decided for each dam and depends on the basic function of the structure.

The design studies for Cherry Valley Dam indicated that maximum seepage through the impervious core might be of the order of 3 to 5 cfs. This is based on the water surface remaining at maximum elevation long enough to saturate the core and establish the phreatic line at its highest position. Since the coefficient of permeability of the core material is about 0.01 feet per day it would take a long time to achieve this condition. The State Fish and Game Commission requires that a minimum streamflow of 5 cfs be maintained downstream from the dam during the fall of year. Obviously it would be very nice if this flow could be maintained by seepage through the dam without the need of a small capacity auxiliary outlet. At any rate, for

Cherry Valley Dam it was decided that the maximum expected seepage would be tolerable.

Upon completion of the embankment, an attempt was made to measure the seepage by means of a weir downstream from the dam but upstream from the channel leading from the outlet valves. Unfortunately this part of the river channel has five to ten feet of semi-pervious material over the bedrock. This material consists mostly of fines washed out of the rockfill by the sluicing during construction. At the weir location the surface flow is concentrated in a well defined channel and it was hoped that the underflow could be cut off without an expensive structure. This was not the case.

That there is underflow is shown by the fact that for a constant reservoir elevation, the amount of flow as measured at the weir varies with the discharge of the outlet valves. As their discharge is increased, the increased depth of flow in the river downstream of the weir raises the water surface in the semi-pervious material and causes increased flow over the weir. When the outlet valves are closed, there is no flow over the weir. At such times none of the seepage reaches the surface of the semi-pervious material.

However, when the discharge of the valves was constant, the flow over the weir varied with the water surface elevation in the reservoir. A record has been kept of the flow over the weir since an increase in this flow with the other variables remaining constant would indicate an increase in seepage through or under the embankment.

Complicating the seepage measurements are the presence of several springs under the downstream portion of the rockfill. These springs were noted during construction. Dye tests were made to find out if there was a connection between them and Cottonwood Creek, which enters the reservoir just above the dam. No connection was found. Since the rockfill is free draining no remedial action was taken. These springs normally only flow during the spring and early summer. Their location has been recorded but it is not possible to separate their flow from the seepage measured at the weir.

There is also a stream gaging station about 3/4 of a mile below the dam. Measurements at this station include the flow of two creeks and several springs between the dam and the station, so that the readings are of little value in determining seepage of the dam.

To date, the seepage has been very small, probably less than 1 cfs. At times when the main outlet valves have been closed it has been necessary to open the small auxiliary outlet in order to maintain the required 5 cfs minimum streamflow.

Settlement

The settlement of a dam embankment is a matter of interest since it is usually considered to be an indication of the degree of compaction achieved during construction. In the case of rockfill, settlement may be taken as measure of the effectiveness of the sluicing in achieving rock-to-rock contact.

The vertical settlement of the points established on the crest of the dam is shown in Figs. 3 to 8 herein. These settlement points are in the core material. They do not show the settlement of the rockfill. The rate of settlement of the core has paralleled the water surface elevations with a time lag of several months. As the reservoir fills, the rate of settlement increases. When the reservoir is drawn down the rate of settlement decreases.

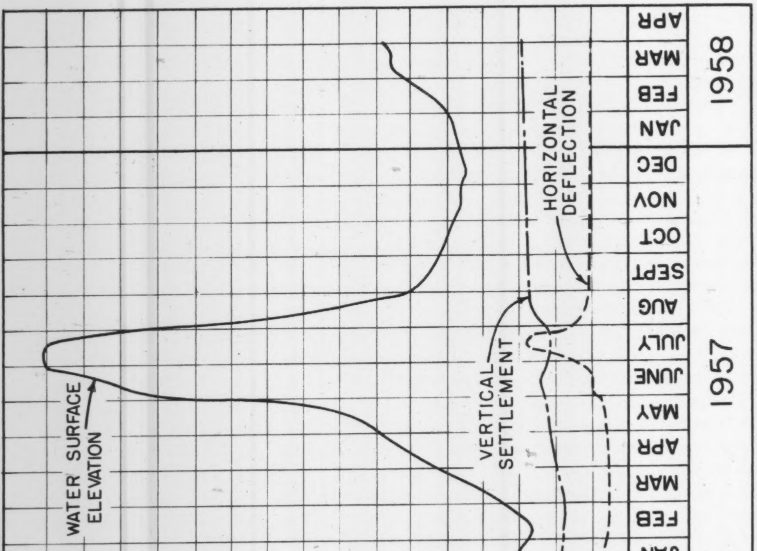
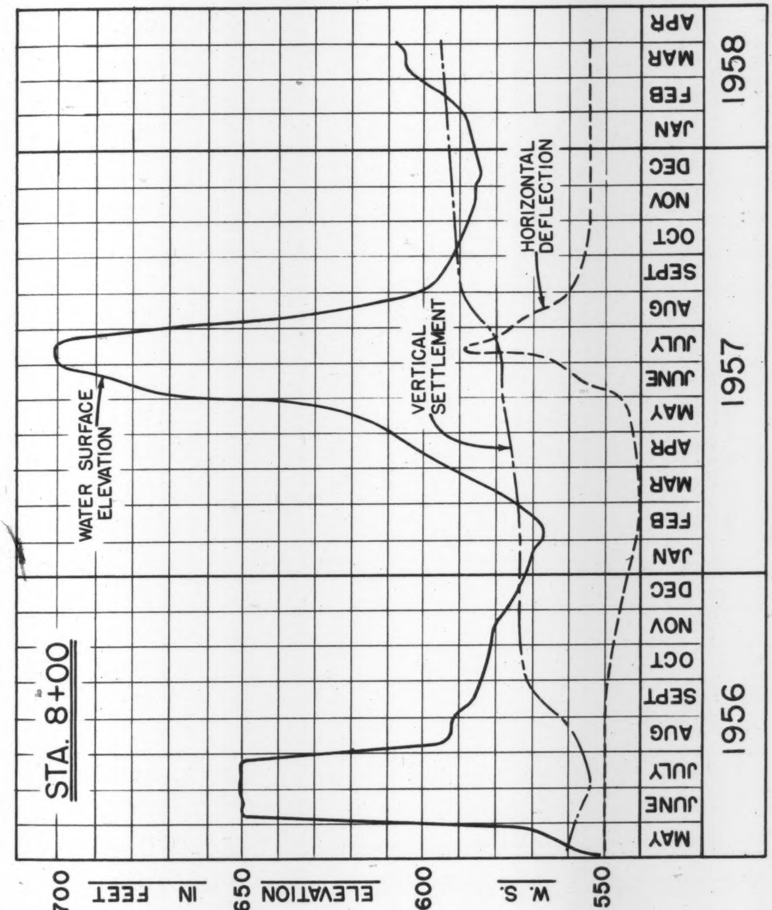

STA. 4+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75'
4700
DEFLECTION
OR
SETTLEMENT
0.50
4650
0.25
4600
0.00
4550
ELEVATION IN FEET
W.S.
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV.
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1956
1957
1958

FIG. 3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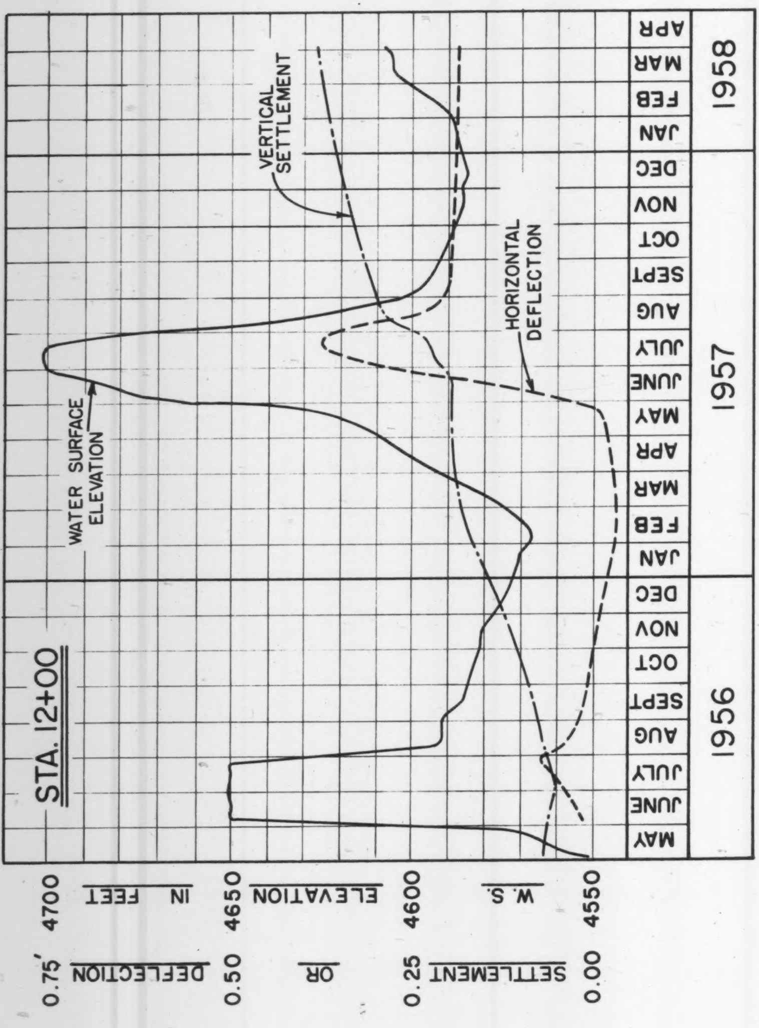
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75' 4700
0.50 4650
0.25 4600
0.00 4550
DEFLECTION OR SETTLEMENT
ELEVATION IN FEET W. S.
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1956
1957
1958

FIG. 4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WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MAR
APR
1956
1957
1958
4700
4650
4600
4550
ELEVATION IN FEET
W.S.
0.75'
0.50
0.25
0.00
DEFLECTION OR SETTLEMENT

FIG. 5

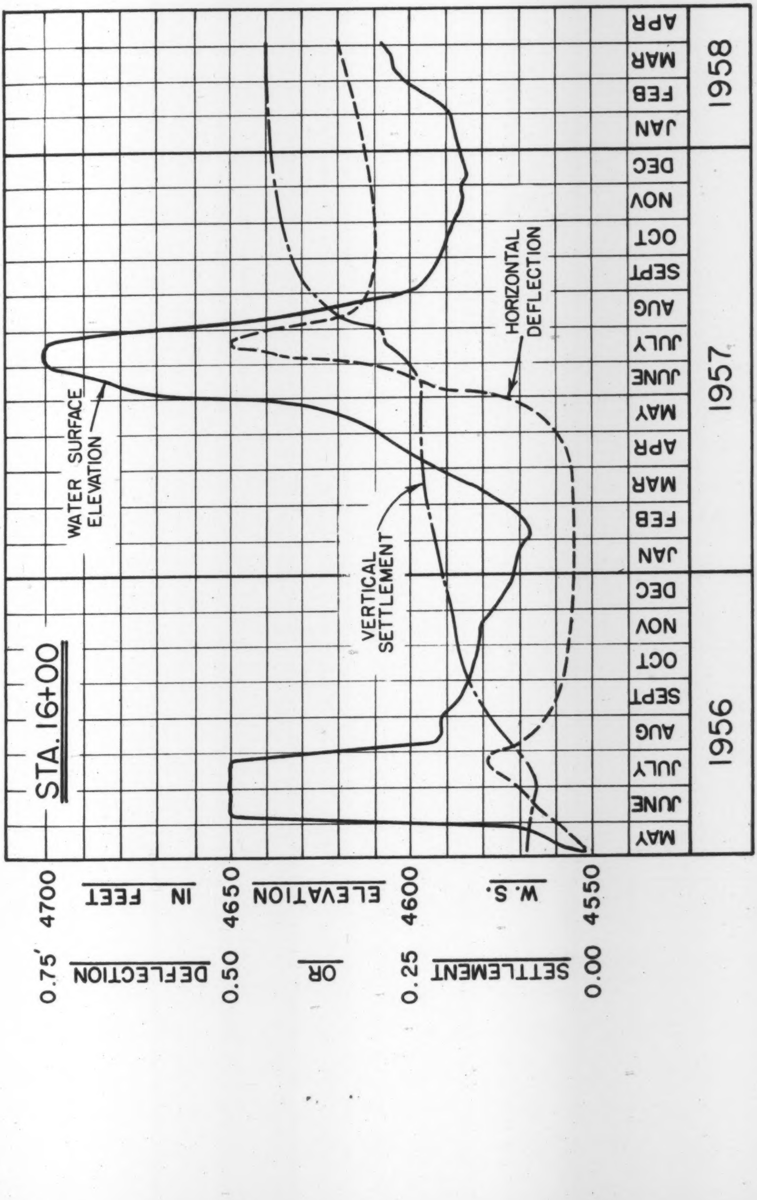
STA. 16+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75' 0.50 0.25 0.00
DEFLECTION OR SETTLEMENT
4700 4650 4600 4550
ELEVATION IN FEET
W. S.
MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR
1956
1957
1958

FIG. 6

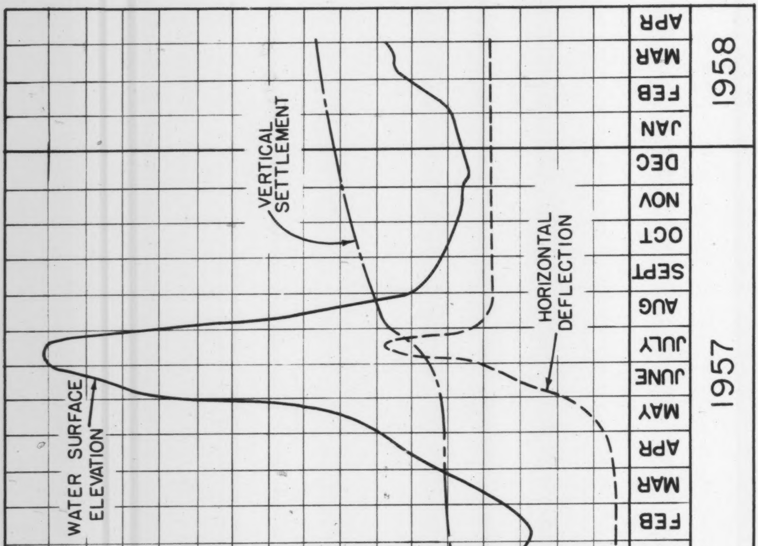
STA. 20+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75' 4700
0.50 4650
0.25 4600
0.00 4550
DEFLECTION OR SETTLEMENT
ELEVATION IN FEET
W. S.
MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC
JAN FEB MAR APR
1956
1957
1958

FIG. 7

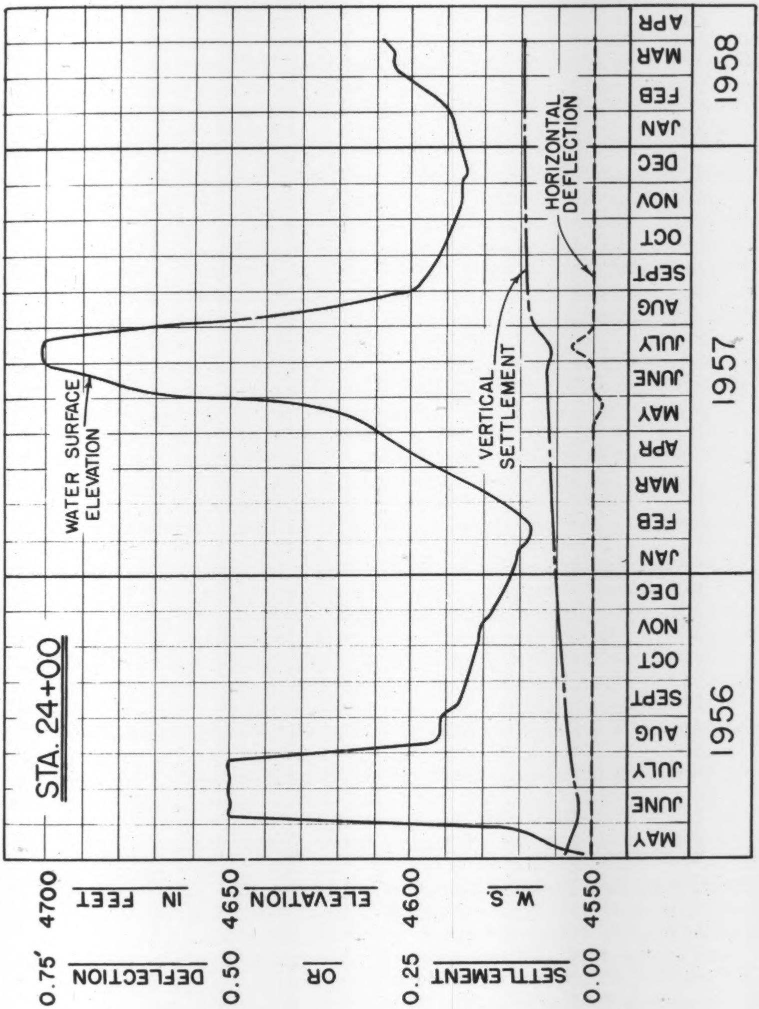
STA. 24+00
WATER SURFACE ELEVATION
VERTICAL SETTLEMENT
HORIZONTAL DEFLECTION
0.75' 4700
0.50 4650
0.25 4600
0.00 4550
DEFLECTION OR SETTLEMENT
ELEVATION IN FEET
W. S.
MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG SEPT OCT NOV DEC JAN FEB MAR APR
1956
1957
1958

FIG. 8

The maximum total vertical settlement to date is 0.45 of a foot, a station 16+00 (the point of maximum embankment height). Of the total settlement, 0.23 of a foot occurred during the first year of operation of the reservoir, and 0.22 of a foot during the second year of operation. A camber of 1% of the vertical height of embankment was built into the crest of the dam. This amounted to 3.3 feet at this point. Thus, in two years operation 13.6% of the camber has been used. Judging from the settlement experience of other embankments, the rate of settlement decreases or ceases with time, the maximum occurring during the first few fillings of the reservoir. It is hoped this will be the case at Cherry Valley since a sway-backed dam is not a thing of beauty.

The settlement at the other points on the dam crest shows the same general pattern as at station 16+00. It is interesting to note that the settlement at each point expressed as a percentage of the embankment height at the point is quite uniform as shown in the following tabulation:

Station	Settlement	Embankment Height	Settlement as % of Embankment Height
4+00	0.10'	80'	0.12
8+00	0.23'	170'	0.13
12+00	0.37'	236'	0.16
16+00	0.45'	323'	0.14
20+00	0.37'	227'	0.16
24+00	0.09'	86'	0.10

As mentioned before, settlement points were not established on the rock fill. Visual observations indicate that the settlement of the rock fill has been greater than the settlement of the core. For example, the berms on the up and downstream faces of the dam were horizontal when built. They are now lower in the central portion of the embankment than at the abutments. The maximum settlement is about 2 feet, measured on the upstream rock berm at elevation 4580. This is probably because the maximum vertical height of rock embankment above either bedrock or the compacted core occurs at this berm.

At the dam crest there is additional evidence of differential settlement between the rock and the core. The crest was constructed to a 40 foot width. The central 20 feet of the crest is the core of the dam, the outer 10 feet on each side is composed of the 2 transition zones plus the rock envelope. The entire crest of the dam is covered with a 6-inch layer of fines and gravel, compacted and bladed to form a roadway for light traffic. After the first filling of the reservoir several longitudinal cracks were noticed along the line of the junction of the core and the transition zone.

These cracks were evidently caused by differential settlement between the rock (including the transition zones) and the core. As the rock settles it has a small component of movement in the upstream (or downstream, as the case maybe) direction. The cracks were not over 1/4 inch wide, and none of them extended continuously more than 30 or 40 feet. There were more of them on the upstream side of the crest than on the downstream. This would be expected since the water action on the upstream rock fill would cause some additional settlement, despite the sluicing performed during construction. The cracks could not be traced more than 10 or 12 inches below the surface. Their locations were recorded and the roadway material was bladed to seal them and prevent entry of water into the fill.

After the second filling of the reservoir these longitudinal cracks were again noted. There was no marked connection between the location of these cracks and the ones in 1956. As before, a record was made of the locations of the cracks and they were filled by blading.

Another indication of this differential settlement is seen at the crest of the dam. The crest was originally constructed with a slight crown in the transverse direction. Due to the increased settlement of the rockfill this crown has increased somewhat, but an accurate measurement of this is not possible since the edge of the fill is quite irregular due to the size of the rocks.

Deflection

A certain amount of movement in a downstream direction is to be expected in any dam embankment as the reservoir is filled. What is not expected is that the embankment move upstream as the reservoir is emptied. Yet that is what has happened at Cherry Valley Dam during its first two fillings. This is depicted graphically on Figs. 3 to 9, on which the dashed line represents the horizontal movement of the survey points (positive in the downstream direction).

Considering station 16+00 again, during the first filling of the reservoir the total movement downstream was 0.14 of a foot. When the reservoir was drawn down, this point moved back upstream again so that just prior to the second filling the residual downstream movement was 0.035 ft. This is a rebound of 0.105 ft. or 72% of the original movement. The rebound was quite rapid at first, closely paralleling the change in water surface.

During the second filling of the reservoir this point moved downstream 0.465 of a foot for a total movement (from empty reservoir) of 0.500 of a foot. (Remember that the first filling of the reservoir was only to elevation 4650, the second filling was to maximum pool at elevation 4700). When the reservoir was drawn down the rebound was 0.195 of a foot, for a residual deflection of 0.305 of a foot. This rebound amounted to 43% of the deflection occurring during the second filling, or 39% of the total deflection from empty reservoir.

The other points moved in a manner similar to that at 16+00 except that during the first drawdown of the reservoir they rebounded to a position upstream from their initial position with an empty reservoir. The amount of such movement at these points was:

Sta. 4+00	0.025 ft.
8+00	0.055 ft.
12+00	0.035 ft.
16+00	none
20+00	0.030 ft.
24+00	0.017 ft.

This phenomenon is hard to explain. The thought immediately occurs that an error was made in establishing the points. However the series of readings made on these points prior to, and during, the first filling of the reservoir show no indication that such was the case. The other idea that has been

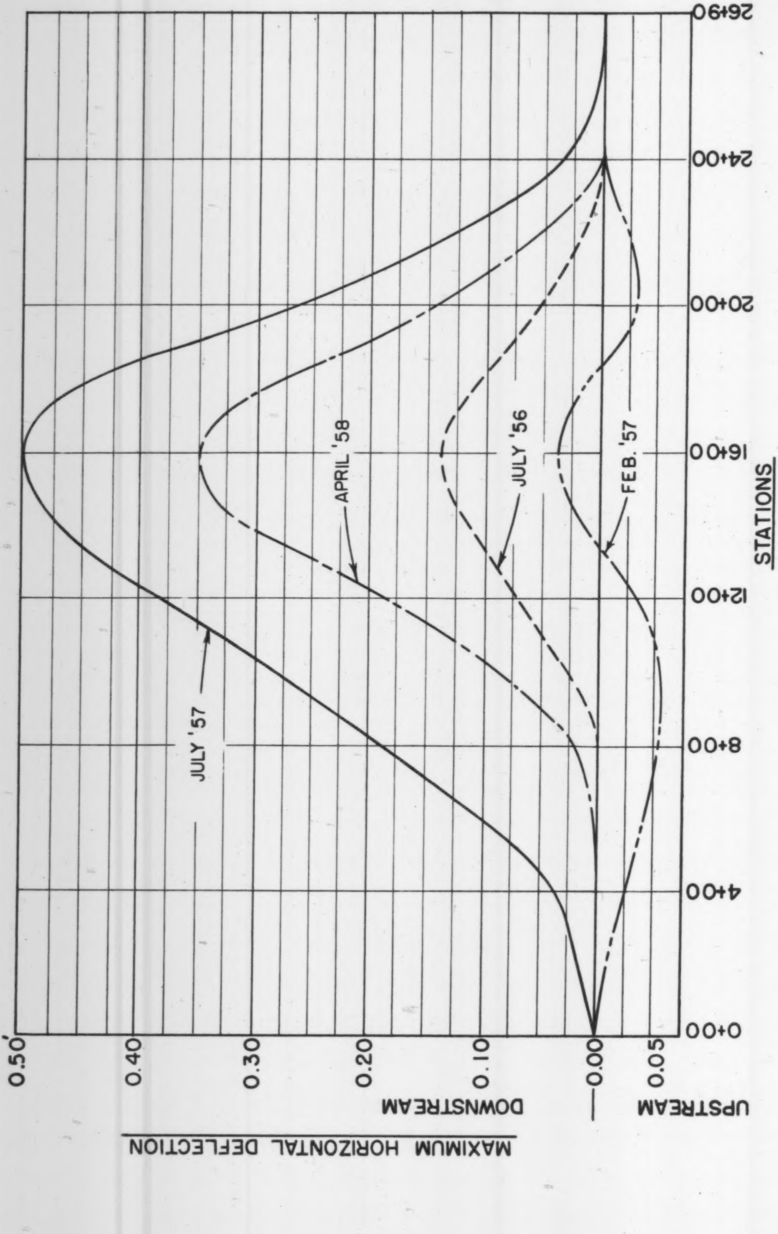
MAXIMUM HORIZONTAL DEFLECTION
UPSTREAM
DOWNSTREAM
0.50'
0.40
0.30
0.20
0.10
0.00
0.05
0+00
4+00
8+00
12+00
16+00
20+00
24+00
26+90
STATIONS
JULY '57
APRIL '58
JULY '56
FEB. '57

FIG. 9

advanced is that due to unequal settlement between the upstream and downstream portions of the embankment there is a net movement in an upstream direction at the crest of the dam. It is understood that similar movements have been observed in other dams and it is hoped that this symposium may produce an answer as to why.

The ratio of the maximum recorded downstream deflection to the height of the embankment at each point is shown in the following table. In the case of the points which have shown a net movement upstream, the amount of such movement has been added to the maximum recorded downstream movement since the total movement during the second filling of the reservoir started from the upstream position of the points. Also shown is the percent recovery at each point as the reservoir was drawn down:

Station	Maximum Deflection "Dm"	Deflection after Drawdown	"Dm" H	Recovery
4+00	0.120 ft.	0.025 ft.	0.0015	79%
8+00	0.245	0.075	0.0014	70%
12+00	0.415	0.220	0.0018	47%
16+00	0.500	0.305	0.0016	39%
20+00	0.295	0.170	0.0013	42%
24+00	0.052	0.017	0.0006	67%

Fig. 9 shows the horizontal deflection at various dates. These curves illustrate quite graphically the movement that has occurred at the crest of the dam as the reservoir has been filled and emptied.

The significance of the dates is as follows:

1. July 1956 Reservoir filled for 1st time (to elev. 4650)
2. February 1957 Reservoir drawn down to elev. 4567
3. July 1957 Reservoir at elev. 4700 (2nd filling)
4. April 1958 Reservoir drawn down to elev. 4608

From the above it is seen that insofar as horizontal movement is concerned the embankment behaves to some extent as an elastic body. As the load increases the horizontal deflection increases and as the load decreases the deflection decreases. These deflections follow the water surface with very little time lag indicating that the embankment adjusts to the increased loads quickly, but not quite as soon as would be the case for a perfectly elastic body.

DISCUSSION

JOSEPH C. DODD,[4] **A. M. ASCE.**—This paper describes a central core rockfill type dam on a rock foundation having a maximum utilization of core material consistent with minimum total embankment volume. Under these circumstances the core will usually be wider than that necessary to hold seepage within the desired limits, as the width is dictated by material availability and economic factors rather than permeability. The use of a drainage blanket under the core downstream of the grout curtain is frequently advantageous in this situation, as the seepage pore pressures affecting the downstream slope stability may be reduced with the lowering of the phreatic line. The blanket assists in relieving temporary pore pressures in the lower portion of the core developed during construction. Also, a large proportion of the seepage crosses the horizontal filter into the drainage blanket, rather than across the stacked filter, as necessary without a blanket. Placement of the horizontal filter is much easier to control than the stacked filter, where the dangers of contamination and variation in thickness are much greater. A reduction in quantity of filter material may be achieved in spite of the greater area with the blanket included, particularly if the phreatic line does not intercept the stacked filter. The thickness of the horizontal filter may be much thinner than the stacked filter because of the method of placement and improved control. The stacked filter cannot be omitted entirely, but a minimum practical width should provide adequate safety against any small hydraulic gradient across the filter resulting from relief of construction pore pressures or variation from the theoretical phreatic line. This was particularly important in the case of the 167' ± high dam being constructed for the Companhia Urbanizadora da Nova Capital at Brasilia in Brazil, where the filter material had to be produced by crushing and processing quarried rock. It was possible to confine the theoretical phreatic line within the core zone by the use of a blanket, thus requiring a much thinner stacked filter than would have otherwise been necessary. The core was made as wide as possible for the desired stability with 2:1 slopes for the outer rock faces, as the very impervious lateritic core material was much more economical than the quarried rock. A slope of 1:1 for the boundary of the core and rock zone was found to be adequate, considering seepage and anticipated construction pore pressures.

The factors favoring a drainage blanket were much greater at Brasilia than in the case of the Cherry Valley Dam, however, as the permeability of the core material was considerably smaller and hence construction pore pressures would dissipate less rapidly, and filter material was much more difficult to obtain. Also, the economics of placement of thin stacked filter zones with greater hand labor would not be as advantageous in this country as elsewhere.

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[4] Civ. Engr., Sverdrup & Parcel, Inc. San Francisco, Calif.

The advisability of a drainage blanket for this type of rockfill dam is, therefore, dependent on many factors peculiar to each situation which must be carefully weighed in terms of cost and performance.

The authors are to be congratulated on the excellent performance of Cherry Valley Dam, as evidenced by the very low values of seepage and settlement during the period since construction.

F. L. LAWTON,[5] F. ASCE.—The authors are to be congratulated on their comprehensive treatment of design, construction and operational features of the 330 feet high central-core Cherry Valley Dam. The basic justification, the economic one, for the selection of the central-core design is clearly given, in the cost of the compacted decomposed granite in place in the core at about one-half the cost of freshly-quarried granite in place in the shoulders.

It would add to the value of the paper if the authors would explain the justification for the use of an earthquake allowance at 0.05g in the analysis of stability of the rock slopes, in view of seismic activity in the general area.

The settlement of the core at the 323 feet high maximum section, at station 16+00, over a two-year period amounts to 0.45 feet or 0.14% of the embankment height. On the other hand settlement of the upstream berm at elevation 4580 amounts to about 2 feet. Even though the rockfill was placed in relatively low lifts of 30 feet, it seems to have been well sluiced. Consequently the differential settlement appears to have been much more significant than the shallow cracks between the core and the transition zone would indicate. Could this have been due to the manner in which transition material was placed during the summer of 1954?

The authors' observations on deflection of the dam under load and the substantial rebound when the reservoir was drawn down are most valuable and unusual. The net upstream movement of the several stations at which measurements were made, except at station 16+00, is indeed hard to explain. Certainly the hypothesis of unequal settlement between the upstream and downstream portions of the embankment causing a net movement in an upstream direction at the crest of the dam is difficult of acceptance. The behaviour of the dam as an imperfectly elastic structure under horizontal loading is also rather unusual in view of the nature of the core material, described as being 95% silty sand, a rather natural result of its origin by deep chemical weathering of granitic bedrock.

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[5] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

H. E. LLOYD,[6] F. ASCE, O. L. MOORE,[7] M. ASCE, AND W. F. GETTS,[8] F. ASCE.—Mr. Dodd's comments on the use of a horizontal drainage blanket under the core downstream of the grout curtain in an impervious core rockfill dam is indicative of the variety of arrangements that may be used with this type of dam. The use of a horizontal drainage blanket under the downstream third of the core was considered for Cherry Valley Dam for the reasons given by Mr. Dodd, namely lowering of the phreatic line in the downstream portion of the core and the possibility of a reduction in the seepage forces acting on the downstream face. However, the core slopes finally used were such that seepage pressures on the downstream slope of the core had no appreciable effect on the theoretical stability. Of more concern to the designers was the question of assuring a tight cut-off along the line between the core and the rock foundation. The length of contact would have been much less had a horizontal drainage blanket been used.

The method of operation of the reservoir also influenced this decision. The water level will not remain at maximum elevation for more than a month or two each year. This is not a sufficient length of time to establish the phreatic line at its maximum theoretical position so that the lowering of the phreatic line is not as important as it might be otherwise. Though this was not considered in the design, it did influence the decision as to the use of a horizontal drainage blanket.

In view of these considerations, it was decided that a horizontal drainage blanket would not be used at Cherry Valley Dam. The authors are in full agreement with Mr. Dodd's statement that the advisability of a horizontal drainage blanket is dependent on the factors peculiar to each situation which must be considered in terms of cost and performance.

Mr. Lawton inquires as to the use of an earthquake allowance of 0.05 g for Cherry Valley Dam. The Cherry Valley region is not subject to violent earthquakes. No recently active faults have been found or are known to exist in the region, and there is no evidence of recent earthquake activity in the vicinity. Inasmuch as the dam is founded upon bedrock of the Sierra batholith, with its great mass of hundreds of cubic miles of dense, sound rock, there is little possibility of large earthquake forces being transmitted to the dam. For these reasons seismic forces were not considered to be significant in the design of the dam, hence the use of a coefficient of 0.05 g.

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[6] Mgr. and Chf. Engr., Hetch Hetch Water Supply, Power and Utilities Engr. Bureau, Public Utilities Comm., City and County of San Francisco, Calif.

[7] Constr. Engr., Hetch Hetchy Water Supply, Power and Utilities Engr. Bureau, City Public Utilities Comm., San Francisco, Calif.

[8] Senior Civ. Engr., Public Utilities Comm., City and County of San Francisco, Calif.

Mr. Lawton also speculates as to whether or not the method in which the transition material was placed during the summer of 1954 has contributed to the differential settlement between the rockfill and the core. The writers believe not, since differential settlement between the rockfill the the core is inherent in a central core rockfill dam. The method of placement used in 1954 gave what amounted to an additional thickness of transition material between the core and the rockfill which would ease this condition somewhat. It is noted that cracking along the top of the dam has been reported at other dams of this type and is not unique to Cherry Valley Dam.

To bring the information in the original paper up to date, additional settlement and deflection data are given herewith:

Date		June 17, 1958	Aug. 14, 1958	Oct. 13, 1958	July 24, 1959
Water Surface Elevation		4686	4695	4575	4530
Sta. 4+ 00	deflection	0.02	0.04	-0.01	0.00
	settlement	0.10	0.10	0.11	0.10
Sta. 8 + 00	deflection	0.08	0.16	0.05	0.06
	settlement	0.22	0.22	0.26	02.8
Sta. 12 + 00	deflection	0.28	0.38	0.25	0.17
	settlement	0.37	0.36	0.43	0.49
Sta. 16 + 00	deflection	0.42	0.54	0.38	0.29
	settlement	0.44	0.44	0.52	0.62
Sta. 20 + 00	deflection	0.20	0.28	0.18	0.10
	settlement	0.36	0.37	0.43	0.50
Sta. 24 00	deflection	0.01	0.02	0.00	0.00
	settlement	0.10	0.10	0.11	0.09

During the 1959 run-off season construction activities on other parts of the project required that the water in the reservoir be kept below elevation 4600. The 1959 run-off in the region was below average which led to early release of the stored water so that at the present writing the reservoir has been drawn down to the lowest point since its original filling. It is interesting to note that as the reservoir was drawn down the crest of the dam moved back upstream past its position at the end of the 1958 drawdown.

As a matter of interest a set of elevations taken July 24, 1959 on the upstream rock berms is presented. Due to the large rock of which the rockfill is composed, it is rather difficult to say what the exact elevation is at any point since adjacent rocks may vary a foot or more in size and elevation. Hence the elevations recorded are an estimate of the average elevation at the points. Conclusions as to the amount of settlement at a point must keep this in mind, also the fact that during construction the same difficulties were present and the berms were constructed to an average elevation by eye rather than to an exact elevation by instrument:

Station	4670	4640	4610	4580	4550
4+00	4670.4				
8+00	4669.5	4639.1	4609.7		
12+00	4669.0	4639.0	4608.5	4579.1	4549.8
16+00	4668.1	4637.5	4607.4	4578.4	4548.2
20+00	4669.1	4639.2	4610.0	4579.0	4550.5
24+00	4669.3	4639.4			

A list of published articles on the construction of Cherry Valley Dam follows:

1. "Bids to be opened August 18 on Cherry Valley Dam Project"
   Construction Illustrated, July 1953, pg. 1
2. "Cofferdam built, contractors to rush fill placement for Cherry Valley Dam"
   Construction Illustrated, June 1954, pg. 5
3. "Cherry Valley Dam job shifts out of high"
   Engineering News-Record, August 26, 1954, pg. 30
4. "Largest hammer drills in action at Cherry Valley Dam"
   Western Construction, September 1954, pg. 57
5. "Equipment geared for big fill job"
   Construction Methods and Equipment, September 1954, pg. 50
6. "Cold storage for Sierra snow"
   Big (published by Goodyear Tire & Rubber Co., Akron 16, Ohio)
   volume 10, number 3, pg. 2
7. "Atkinson completing fill for Cherry Valley Dam; Spillway Outlet Works started"
   Construction Illustrated, September 1955, pg. 1
8. "High, wide and wet"
   Excavating Engineer, November 1954

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3076

ROCKFILL DAMS: PARADELA CONCRETE FACE DAM

By Luis Henrique Gomes Fernandes,[1] Edgard de Oliveira,[2]
and Nuno de Vasconcelos Porto[3]

With Discussion by Messrs. J. Barry Cooke; Armondo da Palma Carlos;
and Luis Henrique Gomes Fernandes, Edgard de Oliveira, and
Nuno de Vasconcelos Porto

SYNOPSIS

The 361-foot high Paradela Concrete Face Rockfill Dam forms part of the hydro-power development of Northern Portugal. The paper discusses the selection of site and type of dam, describes the design and the construction details and procedure, and presents performance data.

GENERAL CONSIDERATIONS

The hydro-electric development of the River Cávado and its tributary the Rabagão, of which the Hidro Eléctrica do Cávado, S.A.R.L. is the concessionary, is situated in the North of Portugal in a district where the drainage area of those rivers has an average annual rainfall of about 2.100 mm (82.7 in.). (See Fig. 1)

Note.—Published essentially as printed here in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1747. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[2] Tech. Sub-Dir., Civ. Engr., Hidro Eléctrica do Cávado, S.A.R.L., Porto, Portugal.

[3] Civ. Engr., Hidro Eléctrica do Cávado, S.A.R.L. Porto, Portugal.

The utilised differences of water level lie between the elevations 903 m. (El. 2,963 ft.) for the Cávado River, 888,7 m. (El. 2,916 ft.) for the Rabagão River and 41 m. (El. 135 ft.), the corresponding gross heads being 862,0 m. (2,828 ft.) and 847,7 m. (2,781 ft.), respectively.

The hydrological system of Portugal, which is practically uniform all over the country with generally low flows in summer time, as well as the scarcity of coal and its poor quality, gave rise to the idea of a development scheme capable of providing a high water storage. Accordingly, a number of storage developments were planned culminating in the Alto Rabagão project, whose big reservoir, in spite of its relatively small drainage area, fully justifies itself economically. In fact, the 781 x 10^6 m3. (633,000 acre. ft.) storage of the Alto Rabagão reservoir provides for a storage in energy of 1.364 x 10^6 kWh. taking into consideration, of course, the heads of the downstream developments. So the Portuguese grid system will have at its disposal a considerable amount of energy in reserve to provide for possible dry years, a fact of greater importance if we remember that the total Portuguese demand in 1964, the year in which the project will be concluded, will be about 4.3 x 10^9 kWh.

The general characteristics of the various projects of this scheme are shown in Table 1.

The Paradela Project

The Paradela project was the fourth carried out by the Hidro Eléctrica do Cávado. (See Fig. 2)

This is a development with a dumped rock-fill dam in the valley of the Cávado creating a reservoir with a capacity of 158 x 10^6 m3. (128,000 acre. ft.) and an area of 380 ha. (939 acres). The water is diverted to an unreinforced concrete lined pressure tunnel which is only steel lined in certain sections, either where there are bad rock characteristics or where the head is greater than 110 m. (361 ft.). The inside diameter of this tunnel varies from 2,80 m. (9.2 ft.) to 2,25 m. (7.4 ft.) and its total length is 9,5 km. (31,170 ft.). Near the reservoir there is a caterpillar gate with 2,50 x 3,80 m. (8.2 x 12.5 ft.) and a steel trash rack operated in an intake shaft. The tunnel intake has a fixed reinforced concrete trash rack.

Near the end of the underground works there is a butterfly valve placed in a chamber, to protect the open air penstock. Upstream from this valve is located a shaft surge tank with compensation galleries and an open air expansion tank.

The penstock layout has a slightly inclined section, amounting to 1.350 m. (4,429 ft.) followed by a 570 m. (1,870 ft.) steeply inclined section. Its diameter varies from 2,35 m. (7.7 ft.) to 2,05 m. (6.7 ft.). It has no expansion joints and its variations in length due to temperature changes are made possible by the distortion of the penstock at the bends; in the straight sections, the tendency to expand or contract is taken up by the internal stress in the pipe.

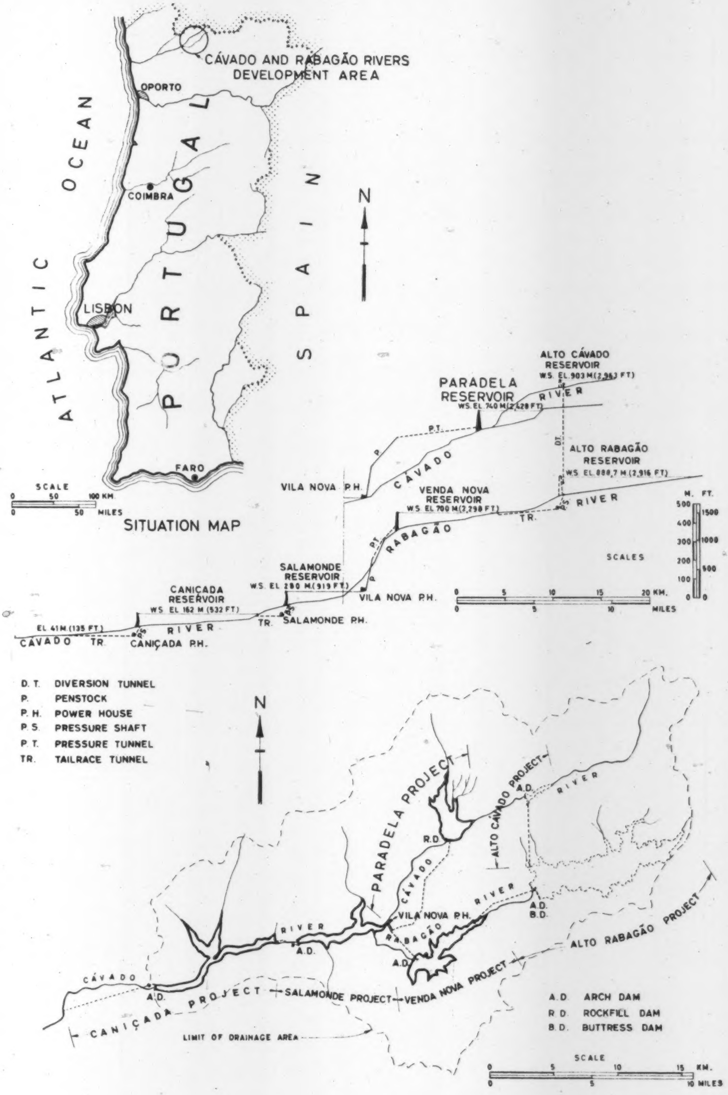

CÁVADO AND RABAGÃO RIVERS DEVELOPMENT AREA
OPORTO
COIMBRA
LISBON
FARO
ATLANTIC OCEAN
PORTUGAL
SPAIN
N
SCALE
0 50 100 KM.
0 50 MILES
SITUATION MAP
ALTO CÁVADO RESERVOIR
W.S. EL. 903 M (2,963 FT)
PARADELA RESERVOIR
W.S. EL. 740 M (2,428 FT)
P.T.
P.
D.T.
RIVER
CÁVADO
ALTO RABAGÃO RESERVOIR
W.S. EL. 888,7 M (2,916 FT)
VILA NOVA P.H.
VENDA NOVA RESERVOIR
W.S. EL. 700 M (2,298 FT)
TR.
P.S.
RIVER
RABAGÃO
P.T.
P.
SALAMONDE RESERVOIR
W.S. EL. 280 M (919 FT)
VILA NOVA P.H.
CANIÇADA RESERVOIR
W.S. EL. 162 M (532 FT)
EL. 41 M (135 FT)
RIVER
TR.
SALAMONDE P.H.
CÁVADO
TR.
CANIÇADA P.H.
SCALES
0 5 10 15 20 KM.
0 5 10 MILES
M. FT.
500 400 300 200 100 0
1500 1000 500 0
D.T. DIVERSION TUNNEL
P. PENSTOCK
P.H. POWER HOUSE
P.S. PRESSURE SHAFT
P.T. PRESSURE TUNNEL
TR. TAILRACE TUNNEL
N
PARADELA PROJECT
ALTO CÁVADO PROJECT
A.D.
RIVER
R.D.
CÁVADO
RIVER
A.D.
B.D.
VILA NOVA P.H.
RABAGÃO
A.D.
RIVER
A.D.
CÁVADO
A.D.
CANIÇADA PROJECT
SALAMONDE PROJECT
VENDA NOVA PROJECT
ALTO RABAGÃO PROJECT
LIMIT OF DRAINAGE AREA
A.D. ARCH DAM
R.D. ROCKFILL DAM
B.D. BUTTRESS DAM
SCALE
0 5 10 15 KM.
0 5 10 MILES

FIG. 1.—MAPS AND PROFILE

TABLE 1.—DATA ON THE CÁVADO-RABAGÃO PROJECTS

Dam	Maximum heighth	Effective capacity	Drainage area	Power House	Static head	Units	Annual output
A - Arch R - Rock-fill B - Buttress	m. (ft.)	10^6 m^3. (acre. ft.)	km^2 (sq. miles)	C - Conventional U - Underground	m. (ft.)	$k\sqrt{A}$ F-Francis P-Pelton	10^6 kWh
Canicada (A)	76 (249.4)	138 (112,000)	783 (302)	Canicada (U)	121 (397)	2x32.000 (F)	260
Salamonde (A)	75 (246.0)	55 (45,000)	623 (241)	Salamonde (U)	127 (417)	2x25.000 (F)	200
Venda Nova (A)	97 (318.3)	92 (75,000)	342 (132)	Vila Nova (C)	414 (1,358)	3x30.000 (P)	200
PARADELA (R)	110 (360.9)	158 (128,000)	126 (49)		460 (1,509)	(2)60.000 (F)	260
(1) Alto Rabagão (A & B)	(103 & 65) (337.9 & 213.3)	776 (629,000)	210 (81)	Alto Rabagão (U)	192,8 (784)	2x40.000 (F)	(3)1.200
(1) Alto Cávado (A)	98 (91.9)	5 (4,000)	102 (39)				

(1)- Proposals requested.
(2)- Can utilise one of the Venda Nova units.
(3)- Dry year support to the Portuguese grid system.

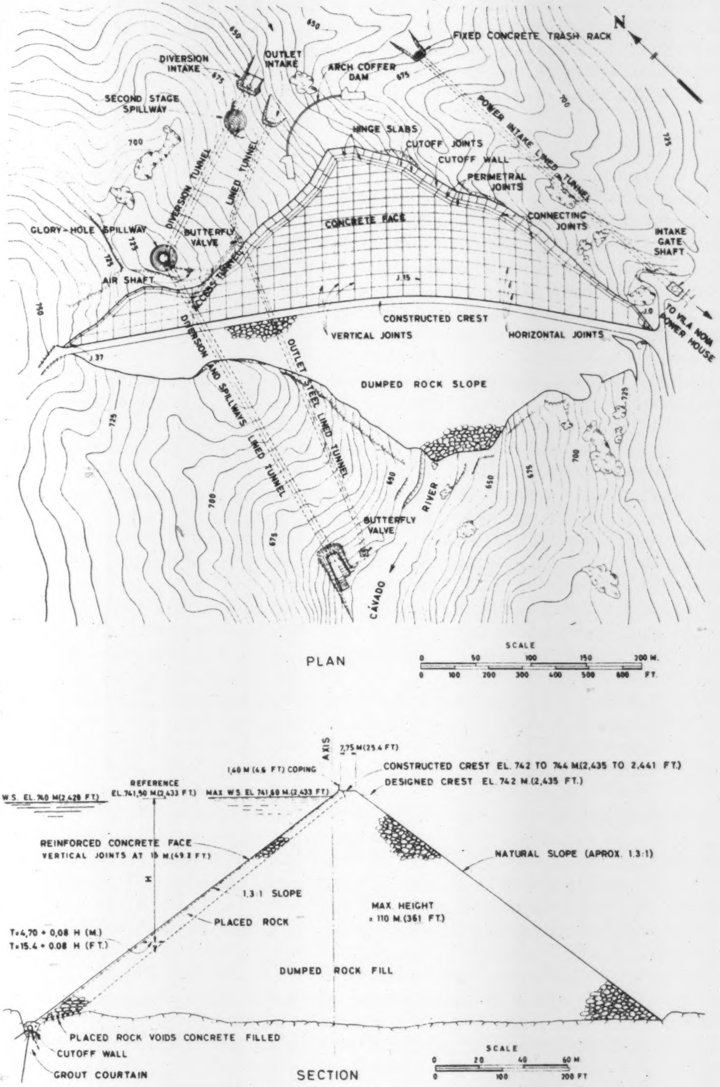

N
FIXED CONCRETE TRASH RACK
OUTLET INTAKE
DIVERSION INTAKE
ARCH COFFER DAM
SECOND STAGE SPILLWAY
POWER INTAKE LINED TUNNEL
HINGE SLABS
CUTOFF JOINTS
CUTOFF WALL
PERIMETRAL JOINTS
DIVERSION TUNNEL
LINED TUNNEL
CONNECTING JOINTS
CONCRETE FACE
GLORY-HOLE SPILLWAY
BUTTERFLY VALVE
INTAKE GATE SHAFT
AIR SHAFT
ACCESS TUNNEL
J.15
J.0
J.37
CONSTRUCTED CREST
VERTICAL JOINTS
HORIZONTAL JOINTS
TO VILA NOVA POWER HOUSE
DIVERSION AND SPILLWAYS LINED TUNNEL
OUTLET STEEL LINED TUNNEL
DUMPED ROCK SLOPE
BUTTERFLY VALVE
RIVER
CÁVADO
650
675
700
725
750
PLAN
SCALE
0 50 100 150 200 M.
0 100 200 300 400 500 600 FT.
AXIS
2,75 M (25.4 FT)
1,40 M (4.6 FT) COPING
CONSTRUCTED CREST EL. 742 TO 744 M.(2,435 TO 2,441 FT.)
DESIGNED CREST EL. 742 M.(2,435 FT.)
REFERENCE EL. 741,50 M (2,433 FT.)
W.S. EL. 740 M (2,428 FT.)
MAX W.S. EL. 741,60 M (2,433 FT.)
REINFORCED CONCRETE FACE
VERTICAL JOINTS AT 15 M.(49.2 FT.)
NATURAL SLOPE (APROX. 1.3:1)
H
1.3:1 SLOPE
MAX HEIGHT = 110 M.(361 FT.)
PLACED ROCK
T=4,70 + 0,08 H (M.)
T=15.4 + 0.08 H (FT.)
DUMPED ROCK FILL
PLACED ROCK VOIDS CONCRETE FILLED
CUTOFF WALL
GROUT COURTAIN
SECTION
SCALE
0 20 40 60 M.
0 100 200 FT

FIG. 2.—PARADELA DAM, PLAN AND SECTION

The water, after passing through an in-line self-closing valve erected in the Vila Nova power station, operates a 117.000 H.P. Francis turbine which has a 50 Htz., 60.000 kVA., generator coupled to its shaft. In case of need the water from the Paradela reservoir can operate one of the three Pelton sets normally operated by the Venda Nova reservoir water (39.000 H.P. turbine and a 30.000 kVA. generator.)

The Paradela Dam

After selecting the stretch of the River Cávado where the dam was to be constructed, its final location was chosen after a comprehensive geological survey carried out to find the most favourable conditions. However, the very varied weathering of the granite, and the numerous faults which exist in many different directions, and often show gouge material of considerable and varying thickness, resulted in any type of rigid dam being out of the question, even those which, like buttress dams, might stand more distortion of the foundations.

Owing to the total absence in this area of earth suitable for the construction of an earth dam, even for a sloping core rock-fill dam, these solutions were also found to be out of the question.

So, the only solution left was a rock-fill dam with reinforced concrete impervious membrane on the upstream face. This was the one finally adopted and constructed. Besides, the fact that there was a good granite quarry about 2 km. from the site, added to the practicability of this solution.

After preparing a first study of the dam, the authors of this paper went on a study visit to the U.S.A. for the purpose of visiting similar projects. Accordingly, they visited in 1954 some of the most interesting projects carried out up to that time in the U.S.A. They take this opportunity of putting on record their gratitude to their American colleagues who gave them such a kind reception and so greatly helped them on their mission.

It is well known that the countless problems raised by the construction of this type of dam cannot be mastered either by calculations or by experiment. And so the plans must be drawn up empirically, based on the teachings of experience acquired either in the execution of, or in observing the behaviour of similar projects already carried out.

The plan of the Paradela dam, with its maximum height of 110 m. (361 ft.), was mainly inspired by those of the Salt Springs—maximum height 100.0 m. (328 ft.)—and the Bear River—maximum height 73,2 m. (240 ft.) dams, both belonging to the Pacific Gas & Electric Company; the required extrapolation being justified by the fully satisfactory behaviour of the former (built 1931), proved chiefly by its many years service.

We would point out here that the maximum height of the dam was lowered from the initially foreseen 112 m. (367.5 ft.) to 110 m. (360.9 ft.), as a result of the glory-hole spillway having been relieved by the construction of a second spillway, which made it possible to reduce the freeboard required for the regularization of the maximum estimated flood.

In its general lines, (Fig. 2), the dam consists of a dumped rock-fill embankment made watertight by an impervious reinforced concrete membrane placed on the upstream face over a layer of placed rock used for a better distribution of the reaction of the dumped rock-fill on the membrane.

The seepage through the foundation is prevented by means of a grout curtain of cement and silicates varying in depth according to the hydrostatic load and the quality of the rock. The connection between this curtain and the watertight membrane covering the face is secured by a concrete cutoff wall poured into an open trench running along the base of the upstream face of the dam.

Foundations

As already mentioned, the site where the Paradela Dam is located has a granite foundation with variable degrees of weathering. By means of extensive excavations—about 370.000 m3. (480,000 cu. yd.)—the foundation area was regularized and the most weathered rock was extracted, although some patches of rather weathered granite were still left.

The following treatment was applied to these patches which might be eventually eroded by leakages from the curtain: After a cleansing by a jet of water, the ground was covered by a layer of hard granite over which the rock-fill would later be dumped. This procedure aimed at a double effect. In the first place it was intended to provide the foundation with a protection layer against soil erosion caused by eventual leakages; this is achieved by the filling in of the voids in the crushed rock layer with the fines carried by the copious sluicing during the dumping of the rock-fill. In the second place, it lessens the settlement of the rock-fill where this is due to the penetrating of rock points or small surfaces into a relatively soft foundation. In fact, the stones will then rest upon a larger surface obtained at the expense of the distribution afforded by the crushed granite cushion.

In the areas where the foundation of weathered granite was sloping and did not allow the cushion granite to remain in position, all the more so because it had to resist the impact of great blocks, a dry rubble wall had to be constructed which would support the steeply sloping crushed granite.

The faults appearing in the surface of the foundation also had to be properly treated, to avoid the erosion of their granite gouge and their more weathered edges by leakages through the banks.

To carry out this treatment the faults and their surroundings were cleaned to a depth of 60 cm. (2 ft.) and the material taken out was replaced by a filter of sand and calibrated crushed granite placed in layers and covered with a layer of "cyclopic" concrete.

As far as the treatment of the foundation is concerned it should be pointed out that there was an area on the left bank, near the downstream foot of the dam, where the granite exists in almost horizontal layers alternatingly hard and weathered. To avoid the washing away of the material in the layers of greatest weathering by waters coming through the banks, a supporting wall of concrete was built with a maximum height of about 10,0 m. (32.8 ft.), which could support a filter made up of various layers of sand and crushed granite to prevent the smallest fines from escaping into the rock-fill.

Location and Shape of the Dam

The location of the dam was fundamentally in accordance with geological requirements because it was decided that the cutoff wall should be located in an area of less weathered rock. Even so, it was necessary to excavate at some places to a depth of 15 m. (49.2 ft.).

In plan, the dam is in the form of a curve. (Fig. 2) This curvature is intended to produce, by means of the arch action, the following results: to oppose the opening of the vertical joints and the tendency that has been verified for the rock-fill to move towards the centre of the valley. Accordingly, the dam is defined by a horizontal directrix, formed by two branches of hyperbolas tangent at their vertices which coincide with joint 15. The hyperbola was chosen because it is a curve concentrating the curvature near its vertex and it was important to have more curvature in the central and lower part of the dam where the distance between the banks is less.

The vertex of the hyperbola was fixed at joint 15 because the greatest settlements were expected at the profile of this joint, for although the profile of the dam is not greater in height on the upstream side, it is in its vertical plane that we find the longest "columns" of rock-fill that will have to transfer the water pressure to the foundation.

In profile (Fig. 2) the upstream face of the placed rock was defined with a uniform inclination of 1,3/1 (1,3 in the horizontal and 1 in the vertical) and the downstream face would have the natural slope of the rock-fill, and this, moreover, was found to be of practically the same value as that assumed above. In consideration of the extremely slight seismic activity of the region and the expected good loose rock-fill dumping, no objection was seen to leaving the downstream face with the natural rock-fill slope, so much so that with the width planned for the crest and with the extra height of 2,00 m. (6.6 ft.) anticipated in the central and highest part of the dam—in expectation of the vertical settlement—the width of the base corresponding to the downstream slope, commonly planned at 1,4/1, was practically attained.

On the other hand, as we expected, the safety of the downstream toe of the dam is increased as a result of the process of construction adopted. In reality the dumping of lifts from a very great height causes a lessening of the slope of the lower part because of the great distance that the big rocks of the rock-fill roll after the fall from such a great height during which they have acquired considerable kinetic energy.

The crest was planned with 7,75 m. (26.4 ft.) width, 0,75 m. (2.5 ft.) being for sidewalk on the upstream side, 6,00 m. (19.7 ft.) for roadway and 1,0 m. (3.3 ft.) for berme on the downstream side.

Following the example of the Bear River Dam No. 1 the profile on the upstream side was not made concave because the distribution of the settlements of the rock-fill under the action of water pressure would provide this advantageous curvature.

The Placed Rock

A great deal of interest was attached to the placed rock layer to be introduced between the loose rockfill mass and the reinforced concrete face on the upstream side. The excellent workmanship available in Portugal made fine results possible in this kind of work. (Figs. 3 and 4).

Considering the height of the dam, it was thought that this layer should be thick, so that the reactions of the loose rock-fill should be well distributed, and of variable thickness increasing with the depth according to the law:

$$e_1 = 3{,}00 + 0{,}05\, h_1 \qquad (e_1,\ h_1 \text{ - meters})$$

or:

$$e_2 = 10 \quad + 0.05\, h_2 \qquad (e_2,\ h_2 \text{ - feet})$$

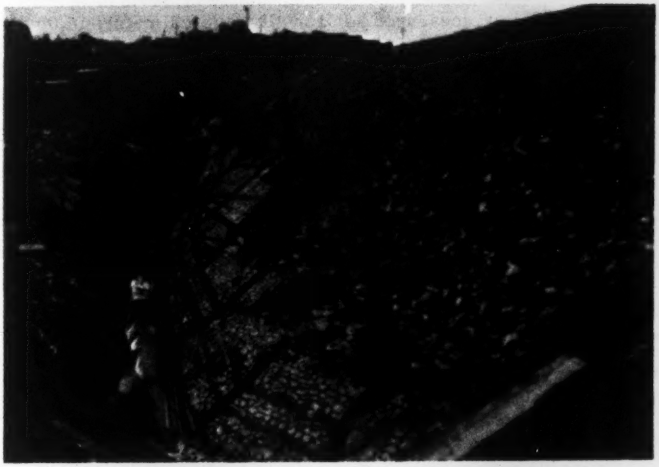


FIG. 3.—CRANES PLACING ROCK (STAGE 1)

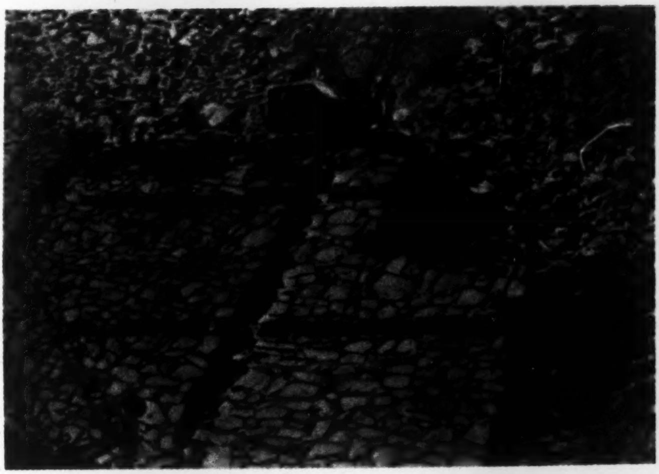


FIG. 4.—CRANE PLACED ROCK DETAIL

in which "h" is the distance to the crest and "e" the thickness measured in the normal to the face. Hence, we have, at the lowest level of the dam, a layer of 8,60 m. (28.2 ft.) or, measured horizontally, approximately 14,00 m. (45.9 ft.).

Where the placed rock-fill comes in contact with the foundation, that is, near the cutoff wall, a special layer, 1,00 m. (3.3 ft.) thick, was made in which the voids between the rocks were filled with concrete. It was intended, by this means, to get a better distribution on the ground of the pressures transmitted by the concrete face while the foundation was at the same time protected, in the neighbourhood of the connection of the concrete face with the cutoff wall, from the erosive action of eventual leakages, the energy of which is not lost by passing through the placed rock. At the same time, it prevents the grout from escaping into the rock-fill in the neighbourhood of the cutoff wall, during the execution of the grout curtain.

The Cutoff Wall

As stated before, the cutoff wall ensures the connection between the grout curtain and the concrete face. Besides, it prevents the leakages which, under the water load, might take place either through the surface on the foundation or through soil immediately below the latter.

Given the characteristics of the foundations, it was thought necessary to provide for an inspection gallery in the body of the cutoff wall along almost the whole of its length. (Fig. 5) It is thus possible to control by means of conveniently placed drains the behaviour of the foundation grout curtain, as well as to carry out repairs to the said curtain, even with a full reservoir. Owing to the very nature of rock-fill dams it would be impossible to carry out such inspections in any other way.

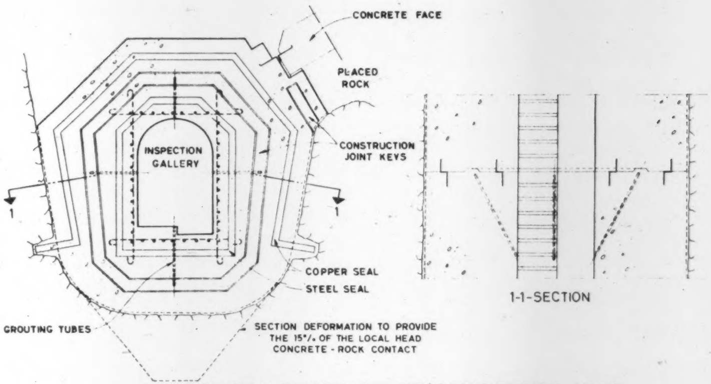
The necessary "gabarit" for the work in the gallery required that its cross-section should have 2,50 m. (8.2 ft.) in height and 1,70 m. (5.6 ft.) in width.

It was also decided that the minimum thickness of the concrete between the gallery walls and the foundation or the reservoir water should vary,according to the elevation, between 1,85 m. (6.1 ft.) and 2,00 m. (6.6 ft.).

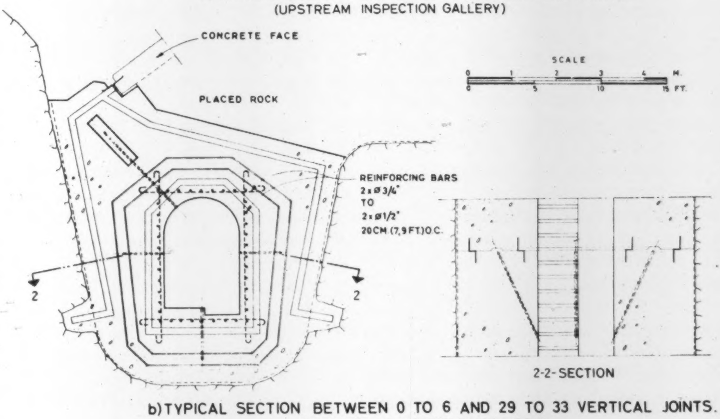
On the other hand, it was determined that the development of the concrete rock contact line should be at least equal to 15% of the head corresponding to the water load in the section, so as to provide a sufficient length for the water percolation.

In the high parts of both banks, and so as to economize on excavations, mainly on the left bank where they reached a depth of 15 m. (49.2 ft.), the location of the cutoff wall was moved downstream and it was constructed over the placed rock. In the final part of the right bank, between joints 33 and 37, the cutoff wall was designed without inspection gallery as it was a part under slight pressure and easily accessible as a result of the normal operation of the reservoir.

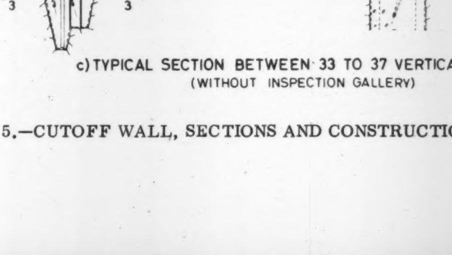
Between consecutive connecting joints of the face the cutoff wall was poured in two blocks to enable the concrete to contract without structural damage. The joints between these blocks, in addition to the connecting keys, were fitted with water seals, made of sheet copper on the outer perimeter and of mild steel sheets on the perimeter of the inspection gallery. Compartments were thus formed which could be filled with cement grout and so watertightness was guaranteed.


CONCRETE FACE
PLACED ROCK
CONSTRUCTION JOINT KEYS
INSPECTION GALLERY
1
1
COPPER SEAL
STEEL SEAL
GROUTING TUBES
SECTION DEFORMATION TO PROVIDE THE 15% OF THE LOCAL HEAD CONCRETE - ROCK CONTACT
1-1-SECTION

a) TYPICAL SECTION BETWEEN 6 TO 29 VERTICAL JOINTS.
(UPSTREAM INSPECTION GALLERY)


CONCRETE FACE
PLACED ROCK
SCALE
0 1 2 3 4 M.
0 5 10 15 FT.
REINFORCING BARS
2 x ø3/4"
TO
2 x ø1/2"
20CM (7,9 FT.) O.C.
2
2
2-2-SECTION

b) TYPICAL SECTION BETWEEN 0 TO 6 AND 29 TO 33 VERTICAL JOINTS.
(DOWNSTREAM INSPECTION GALLERY)


CONCRETE FACE
PLACED ROCK
3-3-SECTION
3
3

c) TYPICAL SECTION BETWEEN 33 TO 37 VERTICAL JOINTS.
(WITHOUT INSPECTION GALLERY)

FIG. 5.—CUTOFF WALL, SECTIONS AND CONSTRUCTION JOINTS

The inspection gallery of the cutoff wall has 3 accesses: at the top of the left bank, at the bottom of the dam, and at the bottom of the col on the right bank. The latter two also allow the drainage of eventual leakages and of the water collected in the inspection drains in the grout curtain.

The access galleries from the banks were concreted in open-air trenches in the foundation and afterwards covered by the rock-fill. The access at the bottom of the valley is through a tunnel open in the hillside.

In the cutoff wall two drains were installed to permit the inspection of the whole of the concrete face. That at the bottom of the dam, close to the access gallery, allows the water contained between the dam and the upstream coffer dam to be emptied while the inflow of the river is diverted through the outlet at the bottom of the reservoir. This drain is constituted by two pipes, 30 cm. (1 ft.) in diameter, each one fitted with two valves in series. The other drain, installed in the lowest point of the right bank col, permits the drainage of the dead volume between the dam and the natural surface; it is fitted with only one pipe, 20 cm. (7.8 in.) in diameter, with two valves in series.

The Impervious Membrane

The impervious membrane of the rock-fill is the delicate point of this type of dam, as it is well known.

In effect, as the rock-fill is subject to large scale settlements, not only during construction as a result of its own weight, but also later on owing to this same weight and the hydrostatic pressure, it becomes necessary to give to the impervious membrane the maximum possible flexibility so that it can follow such large deformations without damage.

The problem is naturally solved when earth facing is employed and, in the case of dams of small and medium height, its solution is comparatively simple if sufficiently elastic materials (such as timber, steel or bituminous concrete) are used. However, it becomes more difficult when to the concrete, either plain or reinforced, is given the role of making the dam watertight, as is the case at Paradela.

Here the flexibility is obtained by dividing the impervious membrane into slabs by means of open joints, some along horizontal lines, others along the slope. The watertightness of these joints is secured by the use of appropriately shaped copper sheets embedded into the concrete of the adjoining slabs.

The impervious membrane of reinforced concrete varies in thickness with the depth, in accordance with the law:

$$e_3 = 0{,}30 + 0{,}00735\,h_3 \qquad (e_3,\ h_3 \text{ - meters})$$

or:

$$e_4 = 1 \quad + 0.00735\,h_4 \qquad (e_4,\ h_4 \text{ - feet})$$

where "h" is the depth in relation to the crest of the dam.

In consequence of this law the impervious membrane has a thickness varying from 0,30 m. (1 ft.) to 1,10 m. (3.6 ft.) measured on the normal to the concrete face, thus guaranteeing a thickness of concrete always greater than 1% of the hydrostatic load, which has been proved, in other enterprises, to be sufficient to prevent the passage of water.

As for the reinforcing steel, the calculation of which is necessarily fallible because it must be based on the computing of relative settlements which cannot be predicted with accuracy, it is also based on the experience acquired in similar enterprises. This experience has shown that a mesh of density corresponding to a section of 0,5% of the concrete section, in both directions (horizontal and in the direction of the slope), is sufficient.

This is how it has been done at Paradela, the reinforcement being placed in one or two layers according to the thickness of the concrete, 1" bars being used at the lower levels (where the concrete is thicker) and 3/4" bars in the upper parts.

As can be seen in Figs. 6, 7, and 8, the Paradela dam has 14 horizontal joints placed at about 10 and 15 m. (32.9 and 49.3 ft.) interval, the shortest being in the lower and central parts of the dam where the greatest settlements are to be expected. The joints along the slope are spaced at 15 m. (49.3 ft.). In the same figures can also be seen details of the different types of joints.

The copper sheet used to seal the joints is 1,5 mm. (0.059 in.) thick in those below elevation 690.50 m. (El. 2,265.5 ft.) and 1,0 mm. (0.039 in.) in the rest. The part of the seal embedded in the concrete varies in dimension according to the joint and its elevation: 274 mm. (10.7 in.) and 224 mm. (8.7 in.), depending on whether situated above or below elevation 690,50 m. (El. 2,265.5 ft.), in the vertical joints;* and, in the horizontal joints, 281 mm. (11 in.) when below that elevation and 231 mm. (9 in.) when above.

The horizontal joints, all of them with 3 cm. (1.2 in.) openings, are filled with timber boards that, owing to their great deformability, absorb part of the deformation of the face which tends to close such joints; in this way an attempt is made to avoid the crushing of the concrete around these joints (a phenomenon observed in some facings already constructed and tried), and, at the same time, the relative rotation of adjoining slabs is made easier without the above mentioned disadvantage.

In order to avoid squeezing the groove of the copper seal when the joint closes, a steam packed cork board is placed against, and glued to, one of its sides. At the other side a strip of "rubberoid" prevents the mortar from entering the space between the seal and the timber, as can be seen in Fig. 6.

In the vertical joints, the opening has 7 or 5 cm. (2.7 or 2 in.) in the central part of the dam where the curvature is greatest, and 2,5 cm. (1 in.) in the lateral and lower parts.

Downstream from the seal the joint opening is filled with steam packed cork board and the groove is protected against squeezing in the same way as in the horizontal joints.

Upstream from the seal the opening widens to 20 and 15 cm. (7.8 and 5.9 in.), respectively above and below elevation 690,50 m. (El. 2,265.5 ft.), and this widening may eventually be filled, in case of need, with a waterproof material to guarantee better watertightness.

The connection between the impervious membrane and the cutoff wall is particularly delicate. In fact, as we have seen, an attempt is made to give the curtain the greatest possible flexibility to allow it to follow the unavoidable settlements of the rock-fill, which reach their maximum in the central section of the dam but slowly decrease in degree. However, the cutoff wall is fixed and practically rigid and, therefore, the impervious membrane may eventually have to withstand important differential deformations near it.

At Salt Springs Dam, during the first few years after construction, numerous cracks appeared in the area near the junction of the face with the cutoff

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*To facilitate description, "vertical joint" means a joint along the slope.

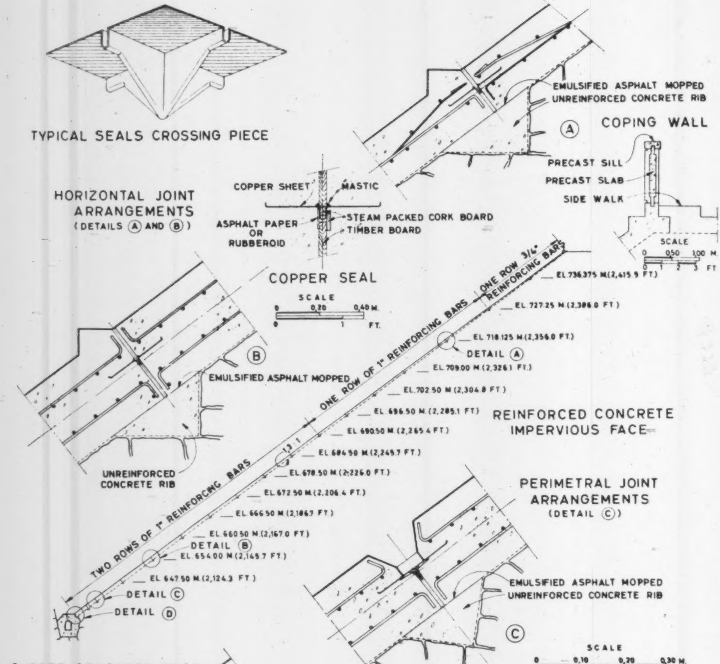
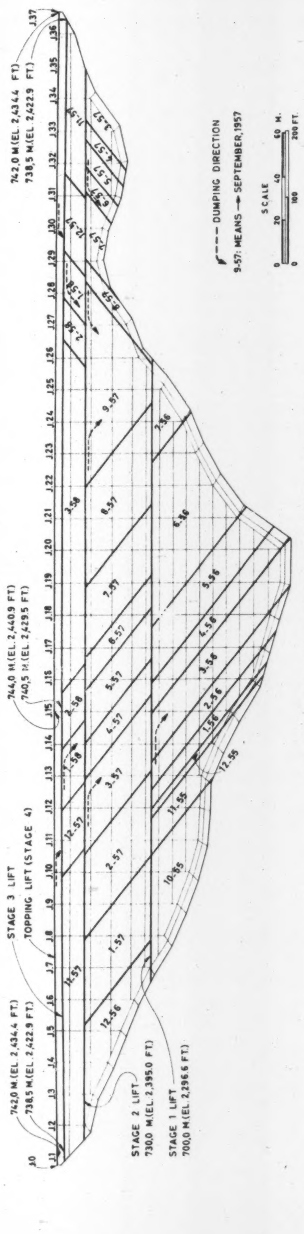
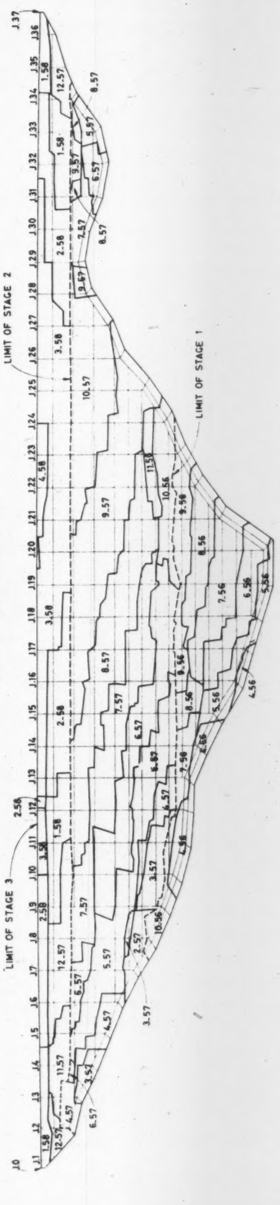

TYPICAL SEALS CROSSING PIECE
EMULSIFIED ASPHALT MOPPED
UNREINFORCED CONCRETE RIB
Ⓐ
COPING WALL
PRECAST SILL
PRECAST SLAB
SIDE WALK
SCALE
0 0.50 1.00 M.
0 1 2 3 FT
HORIZONTAL JOINT ARRANGEMENTS
(DETAILS Ⓐ AND Ⓑ)
COPPER SHEET
MASTIC
STEAM PACKED CORK BOARD
ASPHALT PAPER OR RUBBEROID
TIMBER BOARD
COPPER SEAL
SCALE
0 0.20 0.40 M.
0 1 FT.
ONE ROW 3/4" REINFORCING BARS
ONE ROW OF 1" REINFORCING BARS
TWO ROWS OF 1" REINFORCING BARS
1.3:1
EL. 736.375 M.(2,415.9 FT.)
EL. 727.25 M.(2,386.0 FT.)
EL. 718.125 M.(2,356.0 FT.)
DETAIL Ⓐ
EL. 709.00 M.(2,326.1 FT.)
EL. 702.50 M.(2,304.8 FT.)
EL. 696.50 M.(2,285.1 FT.)
EL. 690.50 M.(2,265.4 FT.)
EL. 684.50 M.(2,245.7 FT.)
EL. 678.50 M.(2,226.0 FT.)
EL. 672.50 M.(2,206.4 FT.)
EL. 666.50 M.(2,186.7 FT.)
EL. 660.50 M.(2,167.0 FT.)
DETAIL Ⓑ
EL. 654.00 M.(2,145.7 FT.)
EL. 647.50 M.(2,124.3 FT.)
DETAIL Ⓒ
DETAIL Ⓓ
Ⓑ
EMULSIFIED ASPHALT MOPPED
UNREINFORCED CONCRETE RIB
REINFORCED CONCRETE IMPERVIOUS FACE
PERIMETRAL JOINT ARRANGEMENTS
(DETAIL Ⓒ)
EMULSIFIED ASPHALT MOPPED
UNREINFORCED CONCRETE RIB
Ⓒ
SCALE
0 0.10 0.20 0.30 M.
0 1 FT.
CUTOFF-CONCRETE FACE JOINT ARRANGEMENTS
DETAIL Ⓓ
S.P. CORK BOARD
Ⓓ
CUTOFF WALL
SCALE FOR JOINT DETAILS
0 0.50 1.00 M.
0 1 2 3 FT.
TIMBER BOARD
STEEL BARS
UNVULCANIZED RUBBER (GUTTATERNA)
FREE SEAL
WELD
GUTTATERNA AND CORK GRANULATE
STEAM PACKED CORK BOARD
ASPHALT PAPER OR RUBBEROID
TIMBER BOARD
VERTICAL AND CONNECTING JOINT ARRANGEMENTS
SCALE
0 0.20 0.40 M.
0 1 FT.
GUTTATERNA IN CONNECTING JOINTS
EMULSIFIED ASPHALT MOPPED
UNREINFORCED CONCRETE RIB
EMULSIFIED ASPHALT MOPPED
UNREINFORCED CONCRETE RIB
ASPHALT PAPER
S.P. CORK BOARD
S.P. CORK BOARD IN VERTICAL JOINTS
TIMBER BOARD IN CONNECTING JOINTS
COPPER SEAL

FIG. 6.—DETAILS OF THE CONCRETE FACE


742,0 M.(EL. 2,434.4 FT.)
738,5 M.(EL. 2,422.9 FT.)
STAGE 3 LIFT
TOPPING LIFT (STAGE 4)
744,0 M.(EL. 2,440.9 FT.)
740,5 M.(EL. 2,429.5 FT.)
742,0 M.(EL. 2,434.4 FT.)
738,5 M.(EL. 2,422.9 FT.)
STAGE 2 LIFT
730,0 M.(EL. 2,395.0 FT.)
STAGE 1 LIFT
700,0 M.(EL. 2,296.6 FT.)
J0
J1
J2
J3
J4
J5
J6
J7
J8
J9
J10
J11
J12
J13
J14
J15
J16
J17
J18
J19
J20
J21
J22
J23
J24
J25
J26
J27
J28
J29
J30
J31
J32
J33
J34
J35
J36
J37
DUMPING DIRECTION
9-57: MEANS → SEPTEMBER, 1957
SCALE
0
20
40
60 M.
0
100
200 FT.

a) PROGRESS OF LOOSE ROCK FILL DUMPING

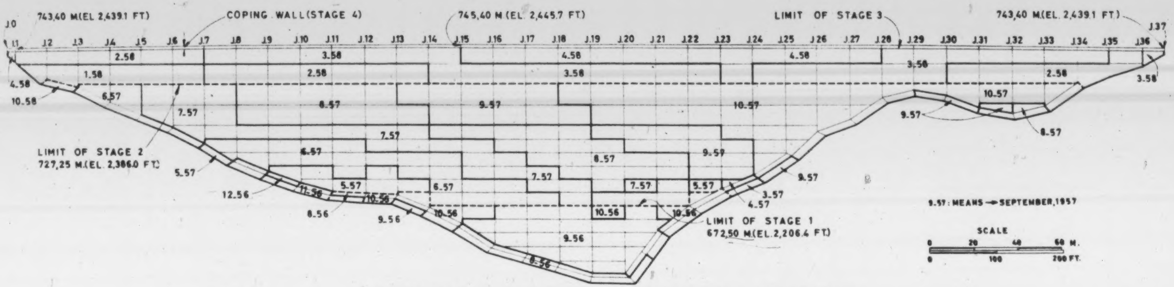
(ON THE UPSTREAM ELEVATION)


LIMIT OF STAGE 3
LIMIT OF STAGE 2
LIMIT OF STAGE 1
J0
J1
J37
2.58
3.58
1.58
4.58
10.57
9.57
8.57
7.57
6.57
5.57
4.57
3.57
2.57
12.57
11.57
11.56
10.56
9.56
8.56
7.56
6.56
5.56
4.56
3.56
2.56

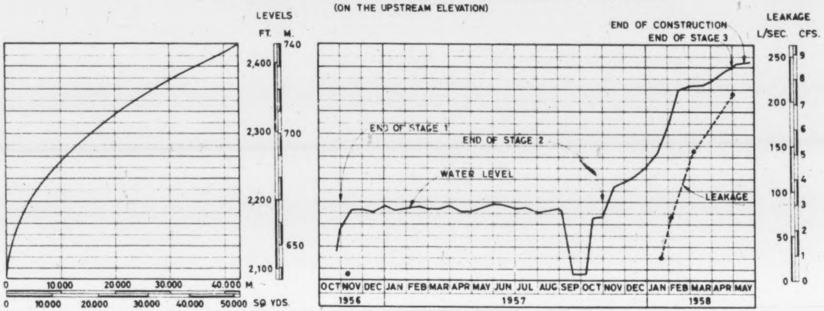
b) PROGRESS OF PLACED ROCK CONSTRUCTION

(ON THE UPSTREAM ELEVATION)

FIG. 7.—PROGRESS OF LOOSE ROCK FILL AND PLACED ROCK


J0
743,40 M(EL 2,439.1 FT)
COPING WALL(STAGE 4)
745,40 M.(EL. 2,445.7 FT)
LIMIT OF STAGE 3
743,40 M(EL. 2,439.1 FT)
J37
J1 J2 J3 J4 J5 J6 J7 J8 J9 J10 J11 J12 J13 J14 J15 J16 J17 J18 J19 J20 J21 J22 J23 J24 J25 J26 J27 J28 J29 J30 J31 J32 J33 J34 J35 J36
2.58
1.58
4.58
10.58
6.57
7.57
3.58
2.58
8.57
7.57
6.57
5.57
4.58
3.58
9.57
7.57
8.57
7.57
10.56
10.57
9.57
5.57
4.58
3.58
2.58
10.57
9.57
8.57
3.58
LIMIT OF STAGE 2
727,25 M.(EL. 2,386.0 FT)
5.57
12.56
11.56
10.56
8.56
9.56
10.56
9.56
9.56
9.57
3.57
4.57
LIMIT OF STAGE 1
672,50 M.(EL. 2,206.4 FT)
9.57 : MEANS → SEPTEMBER,1957
SCALE
0 20 40 60 M.
0 100 200 FT.

a) PROGRESS OF CONSTRUCTION OF CONCRETE FACE SLABS.
(ON THE UPSTREAM ELEVATION)



b) CONCRETE FACE AREA

c) RESERVOIR WATER LEVEL

FIG. 8.—PROGRESS OF CONCRETE FACE, FACE AREA, RESERVOIR WATER LEVEL

wall and more or less parallel to the junction line. This fact was attributed to the excessive rigidity of the face in an area submitted to large differential settlements.

Accordingly, in later projects it was thought to give the curtain an even greater flexibility in that area by the use of perimetric joints parallel to the junction with the cutoff wall, thus forming a sort of articulated frame in which the face is inserted.

At Bear River Dam the use of such a perimetric joint practically stopped those cracks from appearing.

At Paradela the face was constructed with two perimetric joints: one, 4,50 m. (14.8 ft.) away from the junction with the cutoff wall, extends to about elevation 725,00 m. (El. 2,378.7 ft.); the other, at a distance of 3,00 m. (9.8 ft.) from the first, therefore 7,50 m. (24.6 ft.) away from the junction with the cutoff wall, goes up to elevation 695,00 m. (El. 2,280.3 ft.) approximately.

The existence of the large differential settlements referred to, made it necessary that the perimetric joints and the bottom joint with the cutoff wall should be able not only to open and close like the others but also to withstand stronger rotations.

Another phenomenon occurs in some rock-fill dams—and the Paradela Dam will be one of them—that influences the type of joint to be used in the junction with the cutoff wall, and in the perimetric slabs. In fact, in the case of very high dams in comparatively narrow valleys, there appears a tendency for the rock-fill to slide from the banks towards the centre of the valley. This movement of the rock-fill, although partially opposed by the arch action that the curvature of the dam intends to produce, as stated before, may carry with it the impervious membrane and cause a sort of sliding of the latter relatively to the cutoff wall which, being rigid and embedded in the foundation, can be considered fixed, as said. This same sliding can also occur in the perimetric joints.

However the copper seal is bent, it does not easily follow this sliding which can, if it reaches great proportions, lead to the tearing of the seal and the failure of the watertightening system. This problem assumes greater importance in the joint between the impervious membrane and the cutoff wall, where, the latter being fixed, the differential slidings will be greater. In the Bear River Dam, for example, an attempt was made at overcoming this drawback by interlocking the hinge slab and the cutoff wall by means of keys in order to prevent such sliding movements. However, this is equivalent to transferring the problem from the junction with the cutoff wall to the perimetric joint.

At Paradela, in preference to a solution of this type, these joints (the joint at the cutoff wall and the perimetric joints) were designed so as to follow all possible movements, isolated or simultaneous. It is believed that this aim was achieved with the help of a double watertightening system: an appropriately folded copper sheet and, in the event of its failure, a fairly deformable material with great power of adhesion to the concrete and the copper, made with a base of unvulcanised natural rubber ("Guttaterna").

The copper seal, either 1,5 mm. (0.059 in.) or 1 mm. (0.039 in.) in thickness according to whether the joint is below or above the elevation 690,50 m. (El. 2,265.5 ft.), has, as can be seen in Fig. 6, a double groove: one welded to the parts embedded in the concrete, and the other, free inside the first, is in addition covered by the free extremity of the two embedded parts.

Immediately above the copper seal, the opening of the joints is 23 cm. (9 in.) wide and it is filled with "Guttaterna" to a depth of 10 cm. (3.9 in.).

In the event of the first groove of the copper seal being ruptured owing to a serious differential sliding, the second prevents the "Guttaterna" from escaping, under the hydrostatic pressure which reaches more than 10 kg/cm^2. (142.2 psi.) in the lower part of the dam, and as a result of its great plasticity, into the interior of the dam thus leaving an open way for the water.

In order to protect the "Guttaterna" from the heat when the reservoir is empty or partly filled, a board, fixed by small clamps to one of the slabs, runs along the joints leaving an opening of only 5 cm. (2 in.).

The opening of these joints below the copper seal is 3 cm. (1.2 in.) wide and it is filled, as in the horizontal joints, with timber board. As in the remaining joints, the groove of the copper seal is also protected against squeezing by "rubberoid" and steam packed cork board.

In the joint connecting the face to the cutoff wall, and in order to facilitate the movements of the former without crushing the concrete, a sheet of steam packed cork board, 8 cm. (3.1 in.) thick, is placed between the face and the cutoff wall.

The copper seals that make up the watertightening system of the joints must be continuous along all of them. Only in this way can they entirely fulfill their function.

Accordingly, it is easy to understand that the points where any two joints cross give origin to a particularly difficult problem. The continuity of the copper seals meeting at such points must be maintained, regardless of their shapes or dimensions and of the movements which they may be subject to and which may be contrary: one joint opening while the other closes.

In some American dams already constructed (Salt Springs, Dix) the seals of the vertical joints have been kept continuous while, at the meeting points, the seals of the horizontal joints have been flattened and then welded to those of the vertical joints. The different movements liable to occur there were, thus, deliberately ignored, and the copper seal was, in a manner of speaking, abandoned to its own fate.

This was done both in the crossings of the above mentioned joints and in the points where they met the cutoff wall joint or, in the case of the Bear River Dam, the perimetric joints.

At Paradela an attempt was made to solve the problem by means of the specially designed pieces shown in Figs. 6, 9, and 10. The problem is believed to have been satisfactorily solved in this way, because the continuity of the copper seals is secured and the piece has enough deformability to allow it to adapt itself to all movements to be expected.

There is a special piece for each different crossing and it must respect the direction of the joints meeting there. There are even pieces with only 3 branches, such as those used in the crossings with the joint between the concrete face and the cutoff wall.

The Crest

Above the level of the crest and on the upstream side a coping wall, 1,40 m. (4.6 ft.) high was constructed to give the necessary protection against the overtopping of the dam by waves that might originate on the surface of the reservoir.

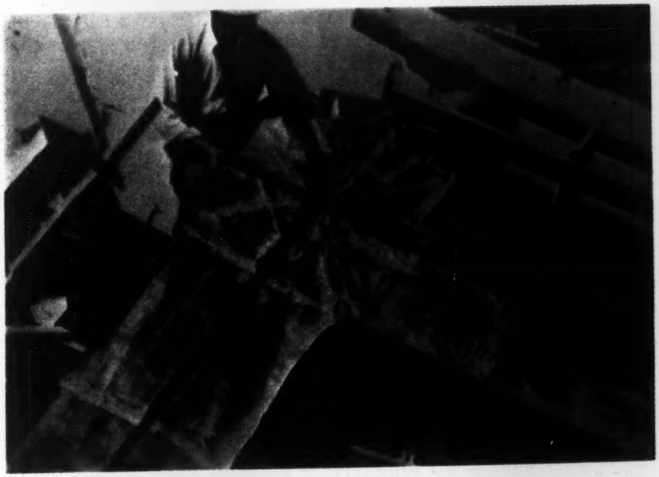


FIG. 9.—COPPER SEALS, DETAIL OF SPECIAL SEALS CROSSING PIECE

FIG. 10.—TYPICAL SEALS CROSSING PIECE

This coping consists of pre-fabricated reinforced concrete slabs, 12 cm. (4.7 in.) thick, fitted in a groove planned for this purpose on the top of the concrete face. The length of these slabs is about 2,3 m. (7.5 ft.), and on the side ends they finish in columns, also of reinforced concrete, joined to the concrete face. Above, the coping is finished with a sill of reinforced concrete, also pre-fabricated. The allowance made in the various grooves and the tarred rope caulking around the slabs make the dam movements possible as well as the eventual readjustments that might be necessary.

General Aspects of the Construction

The Quarry

As has been already mentioned, a granite quarry was used for the construction of the dam. This quarry is about 2 km. (1.2 miles) away in a straight line from the dam, (Figs. 12 and 13) but for the transport of the rock it was necessary to make a road, 4,5 km. (2.8 miles) long with the minimum width of 9,00 m. (29.5 ft.) to provide easy transit for lorries. It must be noted that it proved worthwhile, taking European economics into consideration, to lay an asphalt surfacing so as to reduce the wear and tear of lorry tyres.

The quarry was first worked in two levels: the lower being 40 m. (131 ft.) high and the upper one of heights varying between 60 m. (197 ft.) and 40 m. (131 ft.). At a later stage the 40 m. (131 ft.) high prism of rock, remaining after the previous operations, was worked. It is the opinion of the authors that this type of quarrying with such high banks ought to be avoided owing to the considerable danger involved.

For the placing of the explosives, compressed air drills were used of a type in which the motor itself together with the "bit" goes down the hole which has a diameter of 10 cm. (4 in.), thus making long drills economical without wasting mechanical energy in transmitting it from the opening to the bottom of the hole.

The chief equipment of the quarry was as follows:

Compressors	500 HP
Drilling Machines	10 STENWICK
Shovels	3 LIMA, 3-1/2 cu. yds.
	2 LIMA, 4 cu. yds.
Bulldozers	2 CATERPILLAR D-8

The transport equipment consisted of 22 Euclids of 22 tons, fitted with hydrotarders. During some periods this equipment was reinforced by 5 Cyclops (15 tons).

It may be mentioned here that for the payment of the rockfill a price was fixed per cubic meter, measured in the quarry, including all the operations for extraction, loading, transport, unloading and sluicing. The waste was also paid for at another price per cubic meter dumped in the place destined for it. In this way it was easier to estimate the amount of work to be paid for. The volume of the waste in this dump is measured and the result reduced to rock "in situ", by applying a coefficient corresponding to the rate of volume increase. This coefficient is determined by weighing the material taken from various test pits made in the dump and previous knowledge of the density of the rock.

FIG. 11.—CONCRETE FACE (STAGE 1)

FIG. 12.—CONSTRUCTION OPERATIONS FOR STAGE 2 (NOTICE THE QUARRY IN THE RIGHT BACKGROUND)

FIG. 13.—GENERAL VIEW LOOKING DOWNSTREAM, AT THE END OF STAGE 3 (NOTICE THE GLORY-HOLE SPILLWAY, THE AIR SHAFT AND THE QUARRY)

FIG. 14.—GENERAL VIEW LOOKING UPSTREAM, STAGE 3 LIFT FINISHED (NOTICE ON THE RIGHT BANK THE DEPOSIT OF FINES REMOVED FROM STAGE 1 LIFT, THE PORTAL OF THE SPILLWAY TUNNEL AND THE OUTLET BUTTERFLY VALVE)

In this way the owner will be able to take action without prejudicing the contractor.

Of the total volume of rock extracted, about 20% was waste or considered as such. This percentage exceeded the original estimate and was due not only to geological accidents in the quarry area but also to the rejection of the fines resulting from the blasting of the rock.

This last problem—to separate the fines which are inevitable in the quarrying and to estimate if the percentage they represent is higher than that allowed by the specifications—seems to be the hardest to solve owing to the countless difficulties raised by the continuous supply of material requiring simultaneously such a great number of workmen and so much equipment. The authors are of the opinion that only training and professional consciousness on the part of the excavator operators together with careful supervision, both by those directing the work and the contractor, can solve this problem owing to the difficulty of controlling the way the specifications are being carried out.

The production peak of the quarry with the equipment described reached 10000 tons per 20 hour working day.

In the dumping of the loose rock-fill the only point worthy of note seems to be that the sluicing water used at the Paradela Dam during the dumping was equivalent in volume to four times the volume of the dumped rock, measured in the dam.

To make the sluicing operations easier the monitors were placed at the ends of overhanging booms balanced by a counterweight, the whole forming an easily movable structure on wheels. All movements of the monitor around its vertical and horizontal axes could be remote-controlled.

The arrangements described made it possible to sluice straight on to the dumping from the trucks and this proved to be the most efficient way of removing the fines not separated in the quarry.

As will be described later, very high lifts were employed from relatively narrow tracks about 20 m. (65.6 ft.) wide. This procedure made it possible to make successive partial fillings of the reservoir and resulted in a great compaction of the rock-fill.

Not only did the rocks, when falling from a great height, break their sharp points in the successive impacts but tremendous vibration energy was communicated to the whole when there were extensive slides of rock-fill. These slidings arose from the fact that unstable slopes had been formed in the dumping slope owing to the successive accumulation of piled up rocks. When a dump broke the equilibrium, large masses were set in motion and caused tremendous trepidation, estimated to last about two minutes and sometimes more.

The disadvantage of dumping large volumes of rock-fill from narrow tracks is that of the accumulation of fines that segregate at the very moment the trucks are unloaded.

To get an idea of the depth of the area it would be necessary to take away because it did not guarantee good contact between big rocks without interposing fines, experimental test excavations were made to about 6 m. (19.7 ft.) but concrete conclusions could not really be drawn as observation was very difficult. However, at that depth there seemed to be a greater percentage of big rocks. It was then decided to take away all the fines of a layer 6 m. (19.7 ft.) in height on the upstream side and 2 m. (6.6 ft.) on the downstream side, assuming that it would be preferable to achieve homogeneity in the quality of the dumped rock-fill in order to avoid differential settlements harmful to the concrete face.

This deep treatment was only carried out on the first stage lift at elevation 700 m. (El. 2,297.7 ft.). In the other lifts, as the elevations were greater, a lighter treatment was carried out, the transit layer for the lorries being removed and finally, by monitor jet treatment on the rock-fill, the big rocks on which the later dumpings from other lifts would rest were in this way left exposed.

The Placed Rock

The placed rock layer was constructed with the help of 8 cranes working simultaneously at different levels, and the adequate rock was supplied by dumping from the construction tracks on the lifts (Figs. 3 and 4). The rock utilised was always the largest available that could be handled by the equipment installed; the voids between large rocks were filled up with hand-placed rocks up to 25 kgs. (55 lbs.) in weight. The remaining voids were filled with 5 cm. (2 in.) hard crushed granite.

One problem that required attention was the accumulation of fines in the area of contact between the placed and the loose rock caused by the fines accumulated on the construction lifts (Fig. 14) being carried down when the lorries dumped their supply of rock for the construction of the placed rock layer. With the use of monitors, and even by hand excavation, those fines were removed in order to uncover the large rocks of the loose rock-fill against which the placed rock rested. The supply of rock to each place of work was not without some difficulties chiefly in the areas furthest from the lifts because the rocks reached great velocity and were in the risk of falling along the upstream face of the placed rock or on the part of the impervious face already concreted. With the previous placing of large blocks forming a retaining wall, or by constructing strong timber defences, this problem was overcome and very few large stones went beyond this fixed limit.

It should be further noted that the joints between the rocks on the upstream face of the placed rock were filled with poor cement mortar to prevent the laitance of the concrete of the impervious membrane from escaping during placing.

The Impervious Membrane

The concreting of the face (Figs. 11 and 12) did not present any special problems. The concrete used contained 300 kgs. per cubic meter (505 lbs. cu. yd.), and was made of granite aggregates from the quarry, crushed in an "Almacoa" installation, and mixed in a "Winget" tower with two 3 cu. yds. concrete mixers. The concrete was transported by two radial cable cranes of 6 tons each with two movable and one fixed towers.

The forms of the upstream face were made of sliding panels covering the distance between vertical joints and having about 1,50 m. (4.9 ft.) along the slope. They were supported on guides fitted in the ribs under the vertical joints and on steel beams resting above and below the horizontal joints that limited the slabs being concreted. Manual cable winches pulled up the forms in successive steps, 0,70 m. (2.3 ft.) at a time.

As can be seen in Fig. 6 a step was left in the concrete along the horizontal joints of the membrane. Besides facilitating the access for inspecting or repairing the face, this feature also gave sufficient head to the concrete "liquified" by vibration, for it to get under the sides of the copper seal. Air could escape through little openings made in the forms of the joint, in the part below the groove of the copper seal. To allow the escape of the air which was still retained between the side and the groove of the copper seal, because of its position, some rubber tubes were left with one end in the space to be filled with concrete. When the laitance began flowing from these tubes which were placed at intervals of about 1,5 m. (4.9 ft.), this space was considered to be adequately filled.

As the definite figures are not yet available, it can only be said that an estimate, based on measurements made up to the present over great surfaces of the face, indicates a total consumption, in the face, of a theoretical concrete volume of 24.900 m^3. (32,500 cu. yds.), to which corresponds 7.900 m^3. (10,300 cu. yds.) of concrete beyond the theoretical lines.

The total weight of copper used in the construction, including the seals in the cutoff wall comes to 95 tons.

Construction Schedule

The construction schedule of the Paradela Dam was organised with a view to the possibility of production of energy even in the course of construction.

Accordingly, the work was divided into four stages, as follows:

1st Stage - Dumping of loose rock-fill from a lift 20 m. (65.6 ft.) wide at the elevation 700,00 m. (El. 2,296.6 ft.)
- Construction of the placed rock to elevation 680,00 m. (El. 2,231.0 ft.) (Fig. 3)
- Construction of the reinforced concrete face to elevation 672,50 m. (El. 2,206.4 ft.) (Fig. 11)

During this stage a provisional spillway was provided (Fig. 15) using the same tunnel as the final spillway. Also, later in this stage, the storage capacity of the reservoir was increased by raising the spillway lip by 3,00 m. (9.8 ft.) and by making a provisional timber face up to elevation 678,50 m. (El. 2,226.0 ft.).

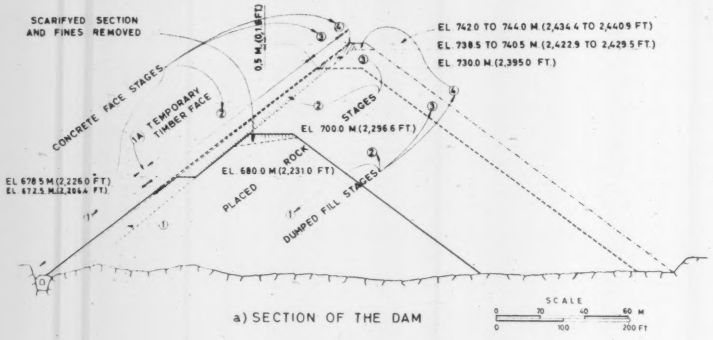
2nd Stage - Dumping of loose rock-fill from a lift 16 m. (52.5 ft.) wide at elevation 730,00 m. (El. 2,395.0 ft.).
- Construction of placed rock and concrete face up to the same elevation. (Fig. 12)

During this stage the final spillway (Fig. 15) was used, some openings having been left in the funnel shape structure with crest at elevation 725,50 m. (El. 2,380.2 ft.).

3rd Stage - Dumping of loose rock-fill from a lift with the theoretical width of the dam at elevation 738,50 m. (El. 2,422.9 ft.), its level varying between this elevation and 740,50 m. (El. 2,429.5 ft.) so as to give the extra height of 2,00 m. (6.6 ft.) designed for the profile across the valley.
- Construction of loose rock-fill to the elevation of the construction crest.
- Construction of the concrete face to an elevation 50 cm. (19.7 in.) below the elevation of the construction crest.

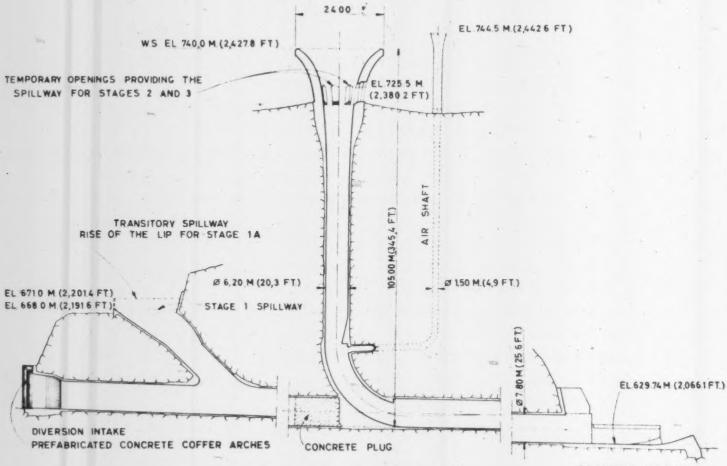
During this stage the same spillway was used as in the 2nd stage.

4th and Final Stage - Dumping from the construction crest of the loose rock-fill prism over the previous stage rock-fill.



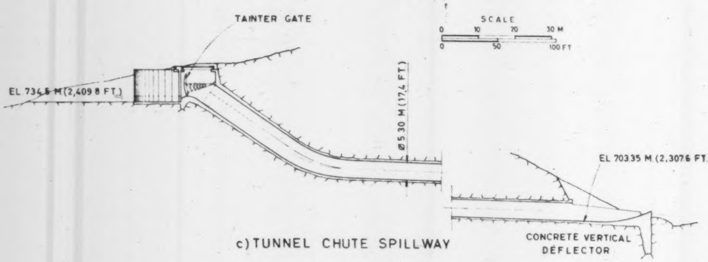
a) SECTION OF THE DAM

SCARIFYED SECTION
AND FINES REMOVED
0.5 M (0,1.6 FT)
EL. 742.0 TO 744.0 M (2,434.4 TO 2,440.9 FT.)
EL. 738.5 TO 740.5 M (2,422.9 TO 2,429.5 FT.)
EL. 730.0 M (2,395.0 FT.)
CONCRETE FACE STAGES
1A TEMPORARY TIMBER FACE
STAGES
EL. 700.0 M (2,296.6 FT.)
ROCK
PLACED
EL. 680.0 M (2,231.0 FT.)
EL. 678.5 M (2,226.0 FT.)
EL. 672.5 M (2,206.4 FT.)
DUMPED FILL STAGES
①
②
③
④
SCALE
0 20 40 60 M
0 100 200 FT



b) GLORY-HOLE SPILLWAY.

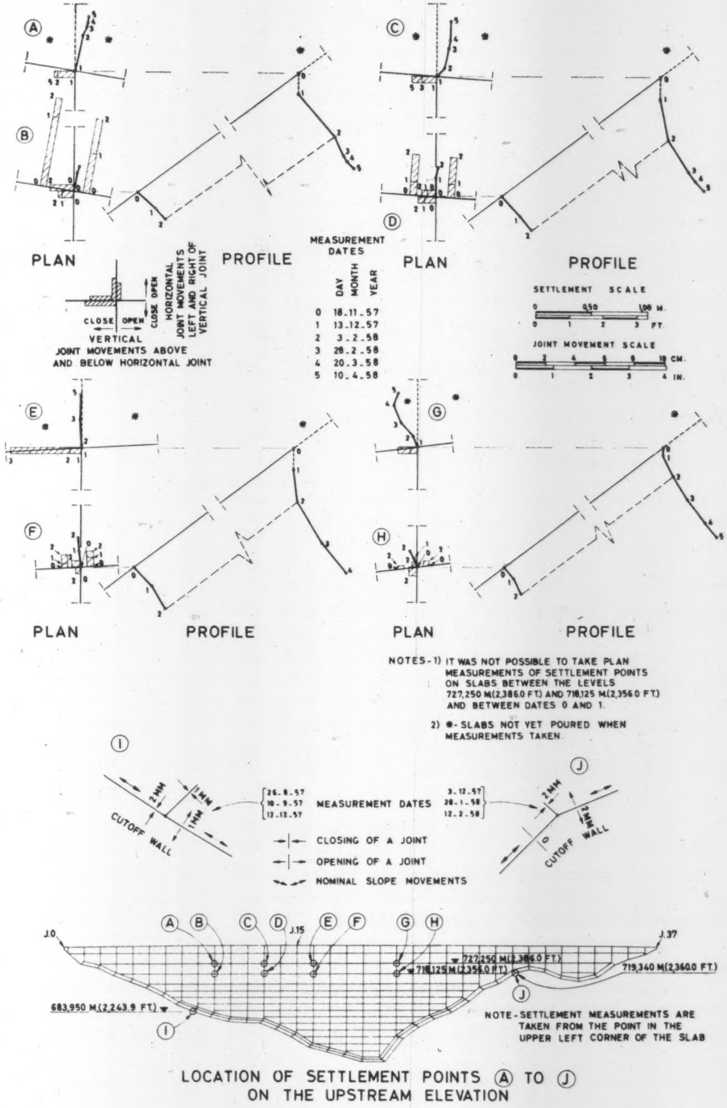
24.00
WS EL. 740.0 M (2,427.8 FT.)
EL. 744.5 M (2,442.6 FT.)
TEMPORARY OPENINGS PROVIDING THE
SPILLWAY FOR STAGES 2 AND 3
EL. 725.5 M
(2,380.2 FT.)
TRANSITORY SPILLWAY
RISE OF THE LIP FOR STAGE 1A
AIR SHAFT
105.00 M (345.4 FT.)
Ø 6.20 M (20.3 FT.)
Ø 1.50 M (4.9 FT.)
EL. 671.0 M (2,201.4 FT.)
EL. 668.0 M (2,191.6 FT.)
STAGE 1 SPILLWAY
Ø 7.80 M (25.6 FT.)
EL. 629.74 M (2,066.1 FT.)
DIVERSION INTAKE
PREFABRICATED CONCRETE COFFER ARCHES
CONCRETE PLUG
CONCRETE VERTICAL
DEFLECTOR



c) TUNNEL CHUTE SPILLWAY

TAINTER GATE
SCALE
0 10 20 30 M
0 50 100 FT
EL. 734.5 M (2,409.8 FT.)
Ø 5.30 M (17.4 FT.)
EL. 703.35 M (2,307.6 FT.)
CONCRETE VERTICAL
DEFLECTOR

FIG. 15.—CONSTRUCTION STAGES, SPILLWAYS


Ⓐ
Ⓑ
Ⓒ
Ⓓ
PLAN
PROFILE
PLAN
PROFILE
CLOSE
OPEN
CLOSE OPEN
VERTICAL
JOINT MOVEMENTS ABOVE AND BELOW HORIZONTAL JOINT
HORIZONTAL JOINT MOVEMENTS LEFT AND RIGHT OF VERTICAL JOINT
SETTLEMENT SCALE
0 0,50 1,00 M.
0 1 2 3 FT.
JOINT MOVEMENT SCALE
0 2 4 6 8 10 CM.
0 1 2 3 4 IN.
Ⓔ
Ⓕ
Ⓖ
Ⓗ
PLAN
PROFILE
PLAN
PROFILE
Ⓘ
Ⓙ
2 MM
1 MM
1 MM
CUTOFF WALL
2 MM
2 MM
CUTOFF WALL
CLOSING OF A JOINT
OPENING OF A JOINT
NOMINAL SLOPE MOVEMENTS
J.0
J.15
J.37
727,250 M(2,386.0 FT.)
718,125 M(2,356.0 FT.)
719,340 M(2,360.0 FT.)
683,950 M(2,243.9 FT.)
LOCATION OF SETTLEMENT POINTS Ⓐ TO Ⓙ ON THE UPSTREAM ELEVATION

MEASUREMENT DATES

	DAY	MONTH	YEAR
0	18	11	57
1	13	12	57
2	3	2	58
3	28	2	58
4	20	3	58
5	10	4	58

NOTES-1) IT WAS NOT POSSIBLE TO TAKE PLAN MEASUREMENTS OF SETTLEMENT POINTS ON SLABS BETWEEN THE LEVELS 727,250 M(2,386.0 FT.) AND 718,125 M(2,356.0 FT.) AND BETWEEN DATES 0 AND 1.

2) *- SLABS NOT YET POURED WHEN MEASUREMENTS TAKEN.

MEASUREMENT DATES Ⓘ: 26.8.57, 10.9.57, 12.12.57; Ⓙ: 3.12.57, 28.1.58, 12.2.58

NOTE-SETTLEMENT MEASUREMENTS ARE TAKEN FROM THE POINT IN THE UPPER LEFT CORNER OF THE SLAB

FIG. 16.—PROGRESSIVE FACE MOVEMENTS

- Construction of the ridge of the concrete face slabs and of the coping wall.
  The openings in the spillway structure were closed; henceforth flood control was done by the glory hole spillway (Figs. 5 and 15), with its crest at the final elevation 740,00 m. (El. 2,427.8 ft.), and by a chute spillway fitted with a radial gate and having its crest at elevation 734,50 m. (El. 2,409.8 ft.).

Energy Output

Besides the value of the reserve represented by the successive storages which were being built up in each different construction stage, the following amounts of energy were produced with the water of the Paradela reservoir in Venda Nova power house:

	Output*	Storage
Utilizing the 1st stage	88.0 x 10^6 kWh	Run-of-river
Utilizing the 2nd stage	62.2 x 10^6 "	110 x 10^6 kWh
Utilizing the 3rd stage	0.5 x 10^6 "	115 x 10^6 "

*Only with Paradela head.

Behaviour of the Dam

Fig. 16 shows some results of observations carried out with a view to determining the movements of the concrete face, and which can be related to the data on the growth of the dam given in Figs. 7 and 8.

Seeing that the dam has been loaded for a very short time and that it has not yet been unloaded, thus allowing further measurements, it is obvious that the elements now given have a somewhat reduced value. Therefore, the publication of further observations to be made, and which it is hoped will be a useful contribution to the greater knowledge of this type of dam, must be left for some future occasion.

However, we would like to point out that the total leakage measured with the level of the reservoir elevation 730,15 m. (El. 2,395.5 ft.) has only reached 209,5 lts/sec. (7.4 cfs.) for a concrete face area, not yet rejoined, of about 34.800 m2. (41,600 sq. yds.).

ACKNOWLEDGMENTS

All the permanent civil engineering works for the construction of the dam were carried out by the contractors, "Sociedade de Empreitadas de Obras Públicas, Ld^a. (SEOP)", of Oporto, Portugal.

The grouting works were conducted by "Sondagens Ródio, Ld^a." of Lisbon, Portugal.

The steel linings, the valves and the gates were supplied and erected by "Sociedades Reunidas de Fabricações Metálicas, Ld^a. (SOREFAME)", of Amadora, Portugal.

DISCUSSION

J. BARRY COOKE,[4] F. ASCE.—The authors have presented a valuable and comprehensive paper on what is presently the world's highest concrete face rockfill dam. They have graciously made references to the influence of the design and performance of Pacific Gas and Electric Company's Salt Springs and Lower Bear River Dams[5] on the design of their Paradela Dam and on the selection of the type of dam. However, the design and construction of P.G. and E.'s Wishon and Courtright Dams proceeded concurrently with that of Paradela Dam, and it is the purpose of this discussion to compare briefly the design and construction of P.G. and E.'s recent concrete face rockfill dams with that of Paradela Dam. The design and construction of the Wishon and Courtright Dams is presented elsewhere.[6]

Foundation and Cutoff

The weathered granite foundation of Paradela is very inferior to the excellent granite bedrock at the sites of the P.G. and E. dams. The use of a concrete bed under the placed rock and the use of the gallery and drains in the cut off wall at Paradela is considered a prudent and logical difference. At Lower Bear River Dam the placed rock was bedded in concrete at some local zones of adverse slope or inferior quality of bedrock to improve effectiveness of grouting, provide insurance against a blowout, and to increase sliding resistance.

Shape of Dam

The writer agrees with the authors' reasons for the moderate curvature in plan and that curvature is important in the central and lower portion of the dam. However, the term "arch action" may not best describe the favorable action of the curvature. The curvature should be such that the resultant settlement normal to the face will always leave a slight upward curvature. More than this minimum curvature in the lower and central area of the face is undesirable since the high water pressures, acting normal to the face, tend to push the face slabs toward the center of the dam. Greater than minimum curvature at and near the crest is desirable for appearance and is satisfactory since water pressures are nominal in the upper portion of the dam. A line across the face will shorten one to several inches due to the "flattening" of the face caused by the settlement. The placed rock lateral contacts are certainly not adequate to even tend to resist such shortening of one to several inches in a several hundred to a thousand foot length by "arch action". What is visualized to occur is a closing of joints due to the distance between adjacent vertical joint lines being shorter after settlement. The amount of each joint closing from this cause may be computed from the differential movement of each slab between joint lines. At the center of the face where both ends of the slab move about the same distance there is no joint closing from this cause. Near the abutment where differential movement is greatest the joint closure due to this cause is greatest. The tendency to close particularly the vertical joints near the abutments favorably tends to offset the opening of those joints by the basic rockfill movements described by the authors later in their paper.

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[4] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

[5] "Rockfill Dams: Salt Springs and Lower Bear Concrete Face Dams," by I. C. Steele and J. B. Cooke, Transactions, ASCE, Vol. 125, Part II, 1960.

[6] "Rockfill Dams: Wishon and Courtright Concrete Face Dams," by J. Barry Cooke, Transactions, ASCE, Vol. 125, Part II, 1960.

The upstream face slope at Wishon, 1.3:1 to 1:1, is steeper than the 1.3:1 slope of Paradela and is economically constructed with the use of "face lifts". Substantial cost savings were made at Wishon and Courtright due to the steep upstream slope and to the use of face lifts. The moderately more conservative upstream face slope at Paradela may in part be due to the authors' not wanting to break other precedents along with the very major precedent they were establishing in maximum height of this type of dam. The downstream slopes of all three dams have resulted essentially in 1.3:1 slopes in the upper portion and 1.4:1 slopes in the lower portion even though layouts and construction lifts were different. This occurs because of the characteristic of rockfills dumped from high lifts to be steeper at the top than near the bottom of the dumped slope, as described by the authors.

The Placed Rock

The placed rock at Paradela is substantially thicker than that at Wishon and Courtright. Comparative thicknesses for Courtright and Paradela are respectively: at top, 7.8 and 10 feet; at 100 feet down, 7.8 and 15 feet and at 290 feet down 11.6 and 24.5 feet. The writer believes that the modern well sluiced and interlocked rockfill of large rocks is as competent as placed rock for upstream slopes of 1:1 and flatter, and that a minimum thickness of placed rock is satisfactory. The major saving at Wishon over previous designs was in the thin placed rock, since placed rock of Wishon Specifications and in the United States costs about 5 times as much as dumped rock. In Portugal the ratio may be much smaller. It would be of interest to know the production rate in cubic yards per 8-hour crane shift at Paradela as well as the above cost ratio. Placed rock also tends to govern the construction schedule unless it is of minimum thickness, and thus adds costs that are not obviously chargeable to the placed rock. The writer wonders if the much thicker placed rock at Paradela might not be due to lower cost in Portugal and the unprecedented height of dam and not to an emphasis on its importance by the designers.

The authors describe the method of placing rock from cranes on the placed rock and the difficulties in supplying the cranes with rock and in removing the fines that accumulate on the surface of the dumped rock as a result of supplying the placed rock from the face of the dumpted rock. Experience at Salt Springs and Lower Bear River was similar; the "face lift" method was adopted at Wishon and Courtright to avoid those troubles, give greater mobility to the cranes and in general to lower the cost of placing rock.

Rockfill

The authors mention "extensive slides of rockfill . . . two minutes . . . etc." The writer has observed surface slides during dumping where the whole dump surface may be in motion in a re-arrangement of surface rock. The dumped slope is very irregular and contains zones of steeper than the average dumped slope. Occasionally a load or a large sliding rock would set off a surface slide. However, from the writer's observations these slides are on the surface and they do not endanger the rear tires of the dump truck. They certainly provide a valuable service in compaction of the fill and they testify to the high shear capability of the rockfill since the energy and vibration do not start a more substantial than surface slide. Does this agree with the nature of slides on the particularly high lifts of Paradela?

Impervious Membrane

The thickness of slab and reinforcing is much the same for all high dams of this type and experience has indicated the accepted design to give satisfactory service and long life. The design features that can vary greatly are joint arrangement, spacing and design. The factors of height of dam and of reservoir operation are of major importance in their influence on the economic spacing and design of joints. The magnitude of movements increases rapidly in the 150 to 350 foot height range. More care in design and higher cost is necessary for joints that may be unwatered only by impairing normal reservoir operation or for joints that may never be unwatered. Wishon reservoir will normally be drawn down to one-half depth each year, but could be completely unwatered in October of any year should it be necessary. Courtright reservoir level will normally be lowered to only 30 feet below full reservoir level each year with dry cycles requiring nearly full unwatering once in perhaps six years. In comparing the joint spacing and details of Paradela and Wishon, the joints at Paradela are more closely spaced, more elaborate and better. The application of the above basic design criteria as well as the lower labor costs at Paradela are believed to be the reasons.

An important consideration regarding the impervious face of this type of dam is that cracks and leakage do not affect safety and, should they occur, the membrane is accessible. If leakage occurs, and represents an economic loss, the trouble can be located and can be repaired under water, or in the dry during a normal or specially scheduled lowering of the reservoir. Since the main settlement and movement of the dam takes place during and shortly after the first filling, repairs made after that time should be permanent, and later leakage is unlikely.

Face Slab Thickness and Reinforcing

The concrete face at Paradela has the same reinforcing as used at Wishon and other high dams of the concrete face type. The theoretical thickness of slab at Paradela of $1 + 0.00735h$ compares with $1 + 0.0063h$ for Wishon. However, the difference in manner of placing rock results in a greater actual average thickness at Wishon. At Paradela the placed rocks are laid with surfaces approximately flush to the plane of the face which reduces the amount of concrete outside the design thickness. The faces at Dix River and Salt Springs were similarly well laid when labor cost in this country was relatively low, and concrete relatively expensive. The economic practice at Wishon was to use large rocks and high capacity cranes and to use minimum hand labor. This results in a stepped and irregular surface and high production per crane shift, that gives minimum cost of placed rock with some penalty in excess face concrete.

Joint Spacing

The vertical joint spacing of 49.3 feet at Paradela is closer than the 60 feet spacing at Wishon. The horizontal joint spacing is 32.9 to 49.3 feet at Paradela and 32.8 to 62.4 feet at Courtright. The top two slabs at Wishon are 77.8 feet long since the broad even profile should not produce differential settlements in the upper portion of the dam and the economy is considered possible. The maximum spacing of joints at Wishon and Courtright is considered important economically to make large pours possible and to minimize the cost of joints.

Joint Ribs

Joint ribs at Wishon were a minimum, the requirement being only a smooth asphalted surface of specified width. This effected economies in simplifying the placing of rock and reducing concrete quantities.

Joint Intersections

The prefabricated "seals crossing piece", Fig. 6 of author's paper, is similar to that successfully used at Lower Bear River Dam No. 1. If the intersecting joints both open, such a crossing is essential to prevent tearing copper. If only one joint opens, or if both joints close, such a crossing is not necessary. At Paradela the 1.3:1 face slope and the dumping of the rock of Stage 3 over Stage 2 tend to produce opening of the upper horizontal joints and measurements of Fig.16 of author's paper indicates horizontal joint openings to have occurred. Also, at Paradela the hinge slabs having a "soft joint" on all sides can move to permit opening of both intersecting joints. The "seals crossing piece" is necessary for many of the joint intersections at Paradela. At Wishon the dumping of the rock and the 1:1 slope at the upper portion of the face was predicted to result in closing of all horizontal joints. The "perimetral" and "connecting" joints at Wishon are cold joints and the U-shaped copper of the perimetral joints is carried through the intersection to permit perimetral joints to open if they choose to. The "seals crossing piece" was considered not to be necessary at Wishon and Courtright and was not used.

Joints

The writer considers the Paradela joints to be particularly well designed. The location of the copper above slab centerline in all joints at Paradela is better than below. Any loading that might develop would be on the bottom edge of the slab and not on the top edge. To have the main reinforcing below the copper and in the thicker portion of the slab that is divided by the copper is fundamentally desirable and costs no more. The important vertical joints are essentially of the same design. The horizontal joints at both Paradela and Wishon are soft joints, which proved very successful at Lower Bear River No. 1, but the Wishon joints are more simple, having a small Z shaped copper, since it was predicted that the Wishon horizontal joints will all close. The use of compressible material under the slab at the cut off joint is considered an improvement and was used at both Paradela and Courtright. A difference in design thinking is that the Paradela perimetral and connecting joints are "soft" joints, whereas at Wishon they are cold joints to provide lines of zero shear and moment and to resist movement, thus minimizing movements of the main slabs. However, the joints are asphalt mopped and do contain U shaped copper with some asphalt filler to permit opening and offsetting. The cold cut off and perimetral joint design is necessary with the simple horizontal joint and the lack of a "seals crossing piece" at intersections, but may have a possibility of crushing. The soft perimetral and connecting joints at Paradela may result in greater joint movements but all the joints are so well designed and constructed that they can in all probability take the movement without trouble.

An engineer reviewing and comparing the Papers of this Symposium on the concurrently designed Paradela and Wishon-Courtright Dams, might conclude that there are substantial differences in the design thinking behind the two dams. It is the writer's belief that the differences are not at all fundamental but are logical products of the differences in foundation conditions, heights, labor costs, reservoir operating schedules and other factors. The unprecedented height of Paradela would certainly have a major influence on design and tend to prevent the departures from previous practice made at Wishon and Courtright. It is hoped that the author's closing discussion will clearly point out any differences in fundamental design consideration, should there be any. Also, it would be of value if the authors could provide crest settlement measurements and other observations taken after the reservoir was filled and unwatered, which occurred since the writing of their paper.

ARMANDO da PALMA CARLOS.[7]—The authors have presented a comprehensive and valuable paper on Paradela Dam, the highest rockfill dam in Portugal. This discussion presents data on the second highest rockfill dam in Portugal, the 207 ft. high Salazar Dam. Salazar Dam, though much smaller than Paradela Dam, is of special interest because of the use of a steel facing as the watertight membrane.

Salazar Dam forms part of the Vale do Sado Lower Course Irrigation Scheme, which also includes the production of hydroelectric power. Full operation began in 1949. The project includes two large storage reservoirs; Pego do Altar Reservoir (Salazar Dam) with a storage capacity of 76,200 acre feet;[8] and Vale de Vaio Reservoir (Trigo de Morais Dam) with a storage capacity of 51,100 acre feet. The Trigo de Morais Dam is a 167 ft. high earth and rockfill dam, watertightness being obtained by means of a central bituminous concrete membrane.

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[7] Head of the Reclamation Div. of the Hydr. Dept., Portuguese Ministry of Public Works.

[8] "Design and Construction of Earthfill and Rockfill Dams and Their Waterproofing Blankets," by Trigo de Morais and Palma Carlos, Fourth Congress on Large Dams.

Main Features of Design

Salazar dam is a rockfill structure provided with a steel facing, laid on a thin concrete layer which covers a layer of masonry; it possesses a cutoff wall with an inspection gallery, from the interior of which the grouting operations of the foundation were carried out. Its main characteristics are:

Maximum height above the cutoff wall foundation	207 ft.
Rockfill structural height	184 ft.
Length of crest	630 ft.
Crest width	16.4 ft.
Slopes:	
Upstream face	1.25 on 1
Downstream face (3 berms—6.6, 8.2 and 9.8 ft. wide)	1.4 on 1 between berms

Geological Features of the Dam Site

The devonian, essentially silicious and argillaceous shales which compose the rock at the dam site are intensively folded and fractured, comprising quartz inclusions in the form of veins and nodules. The low strength of the foundation called for a dam with a wide base.

A large vertical fault, about 13 feet thick, occurs in the river bed, its direction being shown by Fig.17; this fault contains an argillaceous mass with large and small blocks of the early shale. Such geological conditions led to the adoption of the following construction methods for the purpose of avoiding piping:

> At the fault zone, the cutoff wall had to be carried to a depth of (-26.00), which corresponds to half of the hydrostatic head.
>
> Downstream from the cutoff wall, a filter was constructed formed by the following 5 layers, each 1.3 feet thick: fine sand (bottom layer), coarse sand, stonecrusher waste, fine crushed stone, coarse crushed stone (top layer). Thus the voids increased from the bottom to the top layers.

Cutoff Wall, Drainage Gallery and Pumping Well

The cutoff wall is located under the upstream part of the embankment, between the steel facing and the foundation, and contains an inspection gallery. The drainage gallery, embedded in a drain formed by small stones, was constructed in the deepest zone of the foundation, ending in a downstream pumping well. The access to the cutoff wall gallery is made possible by two inspection galleries driven through the valley slopes, Fig. 17.

Rockfill

The cross-section of the dam is shown on Fig. 18. The embankment consists of 485,000 cu. yds. of porphyry rock. The density of the rock was 2.665 and the percentage of voids in the rockfill was 34.4. The largest blocks used in the rockfill weighed 8 to 9 tons.

The rockfill was dumped and partly hand-placed in the central embankment in layers of approximately 9.2 ft. in depth. Cranes were used for placing the large blocks in the upstream embankment, in the downstream toe and in upstream face, as well as for constructing the upstream masonry. This cement mortar masonry is of variable thickness, ranging from 6.6 feet at the bottom to 2.6 feet at the top, its volume being 16,475 cu. yds. The surfacing was made with a concrete layer with an average thickness of 8 in., on which the steel facing sheets were laid. In both upstream and downstream faces corrections had to be introduced to take into consideration the settlements[9] which certainly would occur, not only during the construction but also thereafter, mainly during the filling of the reservoir.

The following corrections were allowed to compensate for estimated settlements occurring after the construction: maximum crown of 2 ft, crest convex in plan, with a deflection of 1.64 ft., and upstream face concave in cross section.

Steel Facing

This type of membrane has been chosen in view of its strength, flexibility and watertightness. Its main features are presented in Fig. 19 and described below:

Steel sheet facing, clamped along the cutoff wall, free on its upper part.

Dimensions of the steel sheets: 24.6 x 8.2 ft; 5/16" thick up to one third of the dam height, 1/4" from there upwards.
Sheets horizontally butt-welded, with 45° chamfered lower edges. Welding seams, at least 3 mm thick, upon butt-plates, 1" x 1/8".

Expansion joints following the maximum slope direction, with semicylindrical butt-plates, 1.3 ft. diameter. Sheets welded to the butt-plates, the edges of which are 60° chamfered; welding seams at least 3 mm thick.

Connections to the cutoff wall by means of 300 mm UNP steel sections, with the lower edges stopping at a distance of 10 mm, from the concrete, this space being filled with plastic bitumen in order to insure complete watertightness; the triangle comprised between the cutoff wall and the membrane was also filled with molten asphaltic concrete. Use of strong gussets plates where the U-shaped structural steel elements change direction, in order to prevent ruptures at such points. Along the beams, anchor bolts 6.6 ft. in length and 1" in diameter, 1 ft. apart from each other.

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[9] "Expected Values and Observed Values in Rockfill Settlements During and After the Construction of Salazar Dam," by Palma Carlos, Fifth Congress on Large Dams.

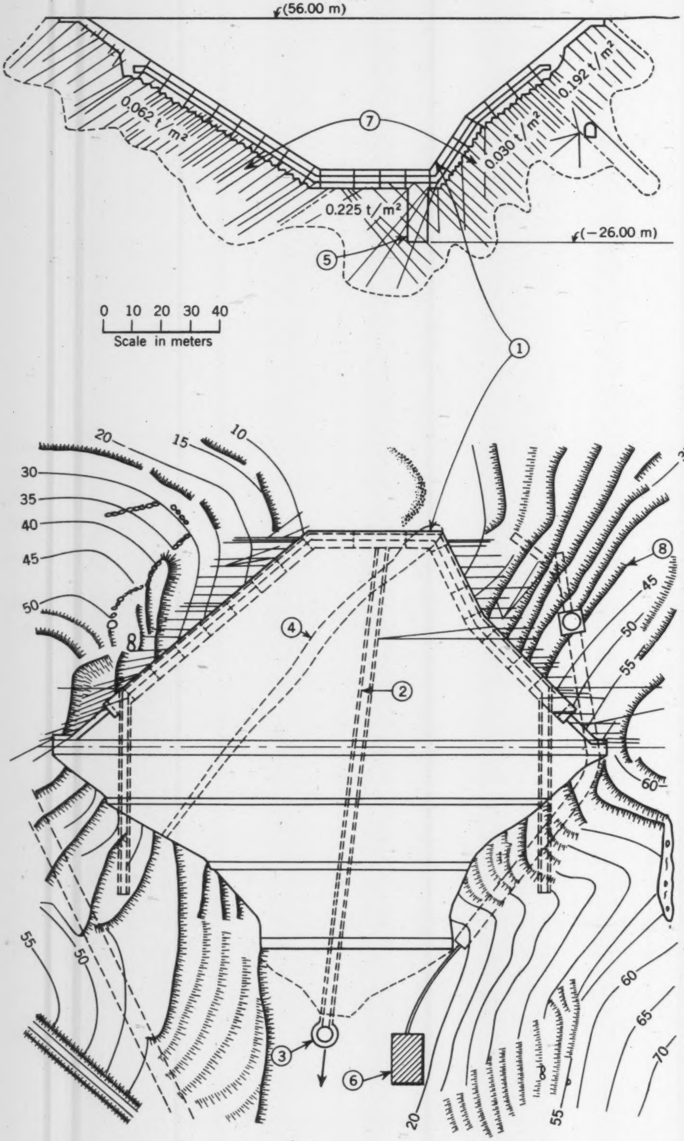
(56.00 m)
0.062 t/m²
⑦
0.192 t/m²
0.030 t/m²
0.225 t/m²
(−26.00 m)
⑤
0 10 20 30 40
Scale in meters
①
20
15
10
30
35
40
45
50
35
⑧
45
50
55
④
②
60
55
50
60
65
70
③
⑥
20
55

FIG. 17.—SALAZAR DAM, FRONT VIEW AND PLAN. (1) CUTOFF WALL. (2) DRAINAGE GALLERY. (3) PUMPING WELL. (4) GEOLOGICAL FAULT WITH A FILTER AT THE DOWNSTREAM SIDE OF THE CUTOFF WALL. (5) DEEP ZONE OF THE CUTOFF WALL AT THE INTERSECTION WITH THE FAULT. (6) POWER PLANT. (7) GROUT CURTAIN. (8) INTAKE TOWER.

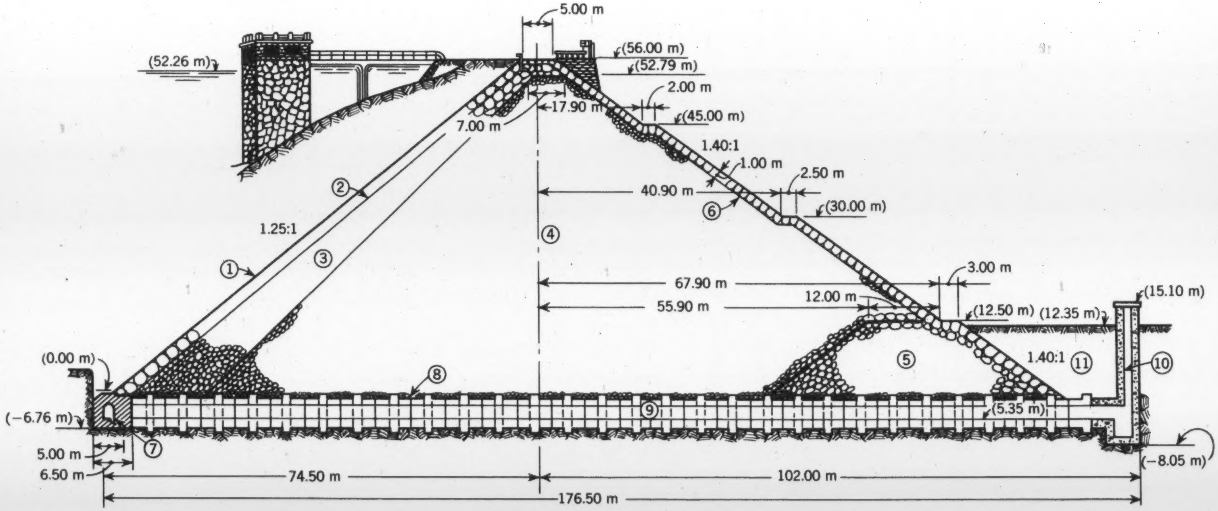
5.00 m
(56.00 m)
(52.79 m)
(52.26 m)
2.00 m
17.90 m
(45.00 m)
7.00 m
1.40:1
1.00 m
2.50 m
40.90 m
(30.00 m)
1.25:1
④
②
⑥
③
①
3.00 m
67.90 m
55.90 m
12.00 m
(12.50 m)
(12.35 m)
(15.10 m)
(0.00 m)
⑤
1.40:1
⑪
⑩
⑧
(−6.76 m)
⑨
(5.35 m)
⑦
5.00 m
6.50 m
(−8.05 m)
74.50 m
102.00 m
176.50 m

FIG. 18.—SALAZAR DAM. CROSS SECTION. (1) STEEL FACING. (2) CEMENT MORTAR MASONRY. (3) UPSTREAM ROCKFILL. (4) CENTRAL EMBANKMENT OF DAM. (5) DOWNSTREAM ROCKFILL TOE. (6) DRY LAID MASONRY. (7) CUTOFF WALL. (8) DRAINAGE GALLERY DRAIN. (9) DRAINAGE GALLERY. (10) PUMPING WELL. (11) TIPPED FILL.

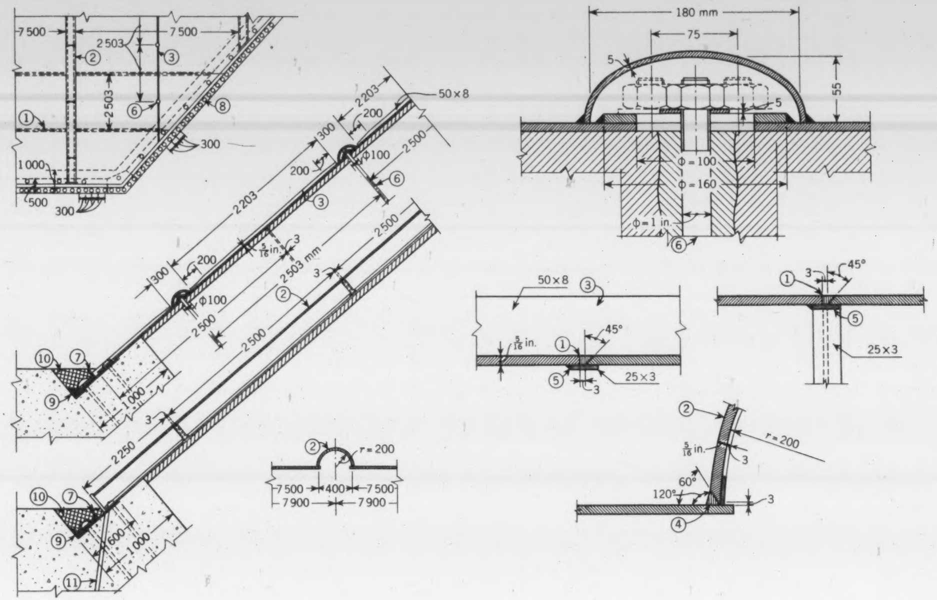
7 500
2 503
7 500
2 503
1 000
500
300
300
50×8
2 203
200
300
Φ100
2 500
2 203
5/16 in.
3
2 503 mm
2 500
2 500
2 250
1 000
600
1 000
r=200
7 500
400
7 500
7 900
7 900
180 mm
75
55
5
5
Φ=100
Φ=160
Φ=1 in.
50×8
45°
5/16 in.
25×3
3
45°
25×3
r=200
5/16 in.
60°
120°
①
②
③
④
⑤
⑥
⑦
⑧
⑨
⑩
⑪

FIG. 19.—SALAZAR DAM, STEEL FACING. (1) HORIZONTAL CONNECTIONS BETWEEN THE SHEETS. (2) BUTT-PLATES OF THE EXPANSION JOINTS. (3) REINFORCING BARS. (4) WELDING OF THE STEEL SHEETS TO THE EXPANSION JOINT BUTT-PLATES. (5) LOWER BUTT STRAPS IN THE HORIZONTAL CONNECTIONS BETWEEN THE PLATES. (6) ANCHOR BOLTS OF THE STEEL SHEETS. (7) UNP STEEL SECTIONS CONNECTING THE STEEL PLATES TO THE CUTOFF WALL. (8) ANCHOR PIECES IN THE CUTOFF WALL. (9) BITUMINOUS JOINT AT THE CONNECTION TO THE CUTOFF WALL. (10) ASPHALTIC CONCRETE. (11) DRAINS CONNECTING WITH THE CUTOFF WALL.

Anchor bolts for the steel sheets, 4.1 ft. in length, 1" in diameter. Provision allowing an expansion up to 3". Expansion nuts and rings covered by protection helmets.

Use of 2" x 5/16" bars between each two anchor bolts, in order to prevent buckling of the steel sheets between those bolts.

Behind the steel facing, drains 2" in diameter connected to the cutoff wall inspection gallery, each one of them connected to an expansion joint. The concrete face was coated with priming coat of Flintkote No. 1 and two waterproofing coats of Flintkote No. 5 before installing the steel face.

Anti-corrosive protection of the steel facing by means of Flintkote bituminous emulsion.

Settlement

During construction the maximum settlement in Salazar dam was 29.9" and it occurred in the downstream face at 60% of the total height of the axial cross section. The more important settlements after the steel facing was installed occurred in the same cross section, their value increasing gradually from bottom to top of dam. After ten years service, settlements measured at crest level were the following: 14.9" on the upstream side and 17.4" on the downstream side. Both values were obtained with full reservoir. Besides these vertical settlements, some horizontal displacements were observed under the same conditions (ten years service and full reservoir). At the crest, the following horizontal displacements were measured: 14.0" on the upstream side and 13.6" on the downstream side.

Leakage

Leakage, either through the steel facing, on account of three fissures having appeared in the welding of the semi-cylindrical butt-plates of the joints, or at the connection of the facing to the cutoff wall, was only observed during the first and the second fillings of the reservoir. Maximum observed leakage was 145.6 U.S. gallons per minute, most of it (79.2 U.S. gallons per minute) resulting from the above mentioned fissures.

The existence of drains, in communication with the cutoff inspection gallery, made it possible to locate immediately the fissures. These were nearly completely sealed with tarp cloth slowly lowered from the crest along the joints. Leakage was in this way greatly reduced even before final repairs by welding. After these repairs, which were done before the third filling of the reservoir, no other leakage was observed through the steel facing, from there on perfectly watertight.

Maintenance

Only now, after ten years service, the reservoir being empty at the end of the irrigation season, was it considered advantageous to apply another coat of protective painting. This repair has already been made this year up to one third of the dam height. It will be completed in the next two years. The total cost of these repairs will be $4,500, extended over a period of three years. Besides this maintenance expenditure, there is another small annual charge with the operation and maintenance of the drainage system.

In rockfill dams watertightness may be advantageously obtained by the use of a steel facing on account of its special strength and elasticity. The initial cost of the steel facing is not considered to be higher than that of the reinforced concrete facing, and the repairs after the first fillings are convenient and of small cost. Long term maintenance cost of a steel facing is moderately small. Drainage behind the facing is very important as it permits the locating easily of any damage in the facing. It appears to be advantageous to protect expansion joints with butt straps. These butt straps will protect the joints against the intrusion of small stones and floating twigs which otherwise might get stuck in the intervals between facing slabs. Experience with the 207 ft. high steel faced Salazar rockfill dam has been most satisfactory in capital cost, performance and in maintenance cost.

LUIS HENRIQUE GOMES FERNANDES,[10] EDGARD DE OLIVEIRA,[11] and NUNO DE VASCONCELOS PORTO.[12]—As this article is being written, the Paradela reservoir is being filled for the third time.[13]

In the two previous fillings the maximum elevations of 738.8 m. (2,424.0 ft) and 740.8 m. (2,430.6 ft) were reached on July 12, 1958 and April 15, 1959 respectively. In the past year (1959) the two spillways worked for the first time.

Although the maximum leakages during the first filling did not exceed 382 litres per second (13.5 cfs) it was decided to empty the reservoir completely at the end of 1958 to carry out a general inspection of the concrete face, and advantage was taken of the occasion to get a general survey of the settlements of the dam. At the same time a campaign of repair was carried out on the damages discovered, although it was not as overall as would have been convenient due to the short time available between the end of the emptying and the start of the rainy season.

The work involved in this repair campaign cost 600,000$00 Escudos ($21,000 in United States currency). The total cost of the dam of 275,000,000$00 Escudos ($9,600,000).

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[10] Tech. Dir., Civ. Engr., Hidro Eléctrica do Cávado, S.A.R.L., Porto, Portugal.

[11] Tech. Sub-Dir., Civ. Engr., Hidro Electrica do Cavado, S.A.R.L., Porto, Portugal.

[12] Civ. Engr., Hidro Electrica do Cavado, S.A.R.L., Porto, Portugal.

[13] In this filling, the level of the reservoir has been maintained around the maximum since the end of December, 1959 until this date (March 10, 1960) and the leakages are on the order of 550 liters per sec. (19.4 cfs.)

During the second filling in 1959 a very marked increase in the leakages was noticed, reaching 3,020 litres per sec (106.6 cfs), and in consequence it was decided to empty the reservoir again completely, to carry out the necessary inspection and eventual repair works. The work executed in that year was done at a cost of 2,000,000$00 Escudos ($70,000).

During the two emptyings, both of which were complete, the position of the bottom outlet on the upstream side of the cofferdam was found to be a great advantage, allowing the flow of the river to pass, and also the advantage was realized of the two small drains not only to allow the flow of the volume of water between the dam and the cofferdam but also the flow of the small leakages beneath the foundations of the cofferdam.

Observations and Work Carried Out Until the End of 1958.—During this year and the first filling, that reached the elevation 738.8 m (2,424.0 ft), the normal maximum level being 740.0 m (2,427.9 ft), the leakages went on rising slowly until they reached a maximum of 382 litres per sec (13.5 cfs). The decrease in the leakages according to the level during the emptying brought nothing special to note.

The various deformations of the dam were measured at several different periods and include: absolute displacements, represented in plan and in profile (the latter in vertical planes parallel to the vertical plane that contains joint 15): relative displacements, of points to both sides of the joints, that measure the movements of the same joints.

The data on the plans refer to:

1) The first observation of the point studied and serves as the origin for future measurements.

2) The observation immediately before the time when the point studied was reached by the water in the first filling (1957-58). June, 1958 was taken for the points not reached by the water.

3) The observation immediately following the unwatering of the point on emptying the reservoir in 1958. September, 1958 was taken for the points not reached by the water.

4) The observation immediately following the unwatering of the point on emptying the reservoir in 1959.

As can be seen[14] in Fig. 20, until the dates of the survey before the first filling, there were already important settlements, principally at the level of the horizontal joint at elevation 727.25 m (2,386 ft), which, in the area of this joint, were mainly the result of the dumping of the rockfill in the third phase of construction.

The filling of the reservoir simultaneous with the construction operations makes a detailed interpretation of the settlements difficult. It is noted that the lateral movements towards the center of the dam are rather more accentuated on the right side than on the left.

After the first filling, an increase in the settlements in plan were noticed, as was expected, as much downstream as towards the center of the dam. It is noted that on the left bank the settlements in plan are very close to the normal of the level lines on the upstream face. This has not taken place on the right side where the settlements have deviated notably towards the center.

Movements upstream were noticed at the points of observation placed on the joint between the perimetral slabs P and Q especially in the area between the connecting joints 12 and 20, as a result of the raising of this joint by the force of compression in the plane of the slabs, which force came from the closing movement of the joints in that area and causing a displacement of those slabs, as shown in Fig. 25 (to be presented subsequently).

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[14] Construction and loading in stages, with some important intervals between measurements on adjacent points, caused discontinuity in settlement lines of upper part of the dam.

In 1958 the maximum displacement downstream was of the order of 0.926 m (3.04 ft) and the maximum lateral displacements towards the center of the dam and measured normally to the vertical planes of the joints, were of 0.152 m (0.50 ft) and 0.291 m (0.95 ft) respectively on the left side and right side.

Until the date of the observations before the first filling there were also important deformations in profile, as can be seen[14] in Fig. 21, chiefly at the level of the horizontal joints at elevations 727.25 m (2,386.0 ft) and 718.125 m (2,356.0 ft), the greater part of which, as has been stated, can be attributed to the dumping of the rockfill in the 3rd phase, and the smaller part to the hydrostatic pressure already being applied to the lower elevation, because the reservoir was in the process of filling as conditions allowed.

It is explained that the dam had already settled when the slabs of the rows A and B were poured. That is, the lay out position of the joint between slabs A and B had already settled considerably, for the reasons previously explained, when the placed rock, corresponding to the rows of slabs B and A, was laid. The laying of the placed rock in the area of the B slabs was done so as to connect the joint B-C in its deformed position with the designed position of the joint A-B. The placed rock of the area of the slabs A was then constructed in the designed position. Although some settlements had taken place up to the time of the pouring of the slabs for these two rows, the designed thickness of the concrete was maintained without an attempt being made, at the expense of excessive thickness, to reach the position of the upstream face that was initially designed.

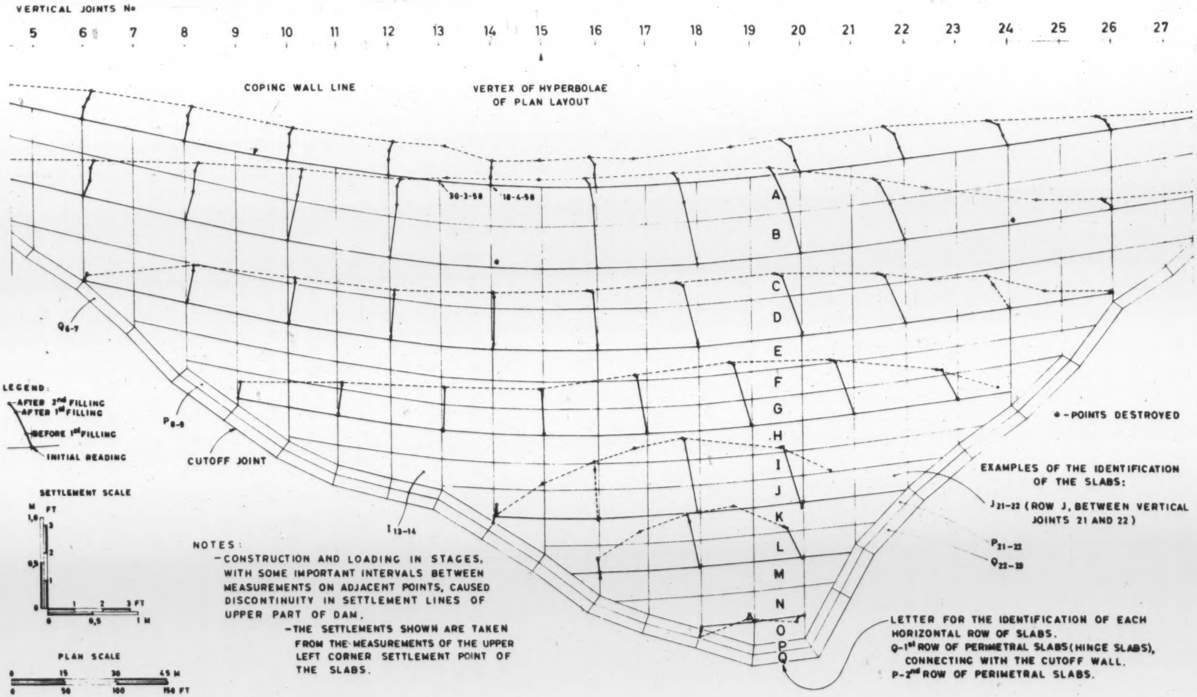
In their general lines these settlements, as well as those resulting from the application of hydrostatic pressure up to elevation 738,80 m (2,424.0 ft), were quite near normal to the profiles of the face in planes parallel to the vertical plane containing joint 15.

In the joint between perimetral slabs (the area between the connecting joints 12 and 20) relative uplifting movements were noticed, detaching the slabs from the placed rock on which they were poured (Fig. 24). The maximum vertical settlement measured to this time was 1.585 m (5.20 ft).

Before the first filling the greater part of the horizontal joints did not show appreciable movement. Only some of the joints of the area constructed in the first phase—to allow the exploitation of the run of the river--showed important closing, with a maximum of 1.6 cm (0.05 ft).

At that time, resulting from the effects of the dumping of the loose rockfill in the third phase of construction, the horizontal joints between slabs A to E, especially in the stretches to the left of vertical joint 16, showed opening movements, particularly in joint C-D between the vertical joints 7 and 13. For example, the maximum opening of 7.0 cm (0.23 ft) was observed in joint C-D, while in D-E (in the same area) the maximum opening was 2.5 cm (0.08 ft). After the first filling the general movement was a closing one, with increases that brought the maximum closing to about 2.3 cm (0.08 ft).

The joints between slabs from A to E, that had opened before the first filling, also showed noticeable closing, which in the cases previously mentioned brought the final positions to an opening of 2.2 cm (0.07 ft) and a closing of 0.2 cm (0.007 ft), respectively at joints C-D and D-E (Fig. 22(a)).



VERTICAL JOINTS No
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
COPING WALL LINE
VERTEX OF HYPERBOLAE OF PLAN LAYOUT
30-3-58
18-4-58
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O
P
Q
Q_{6-7}
P_{8-9}
CUTOFF JOINT
I_{13-14}
LEGEND:
AFTER 2nd FILLING
AFTER 1st FILLING
BEFORE 1st FILLING
INITIAL READING
SETTLEMENT SCALE
M
FT
1,0
3
2
0,5
1
0
1
2
3 FT
0
0,5
1 M
PLAN SCALE
0
15
30
45 M
0
50
100
150 FT
● - POINTS DESTROYED
EXAMPLES OF THE IDENTIFICATION OF THE SLABS:
J_{21-22} (ROW J, BETWEEN VERTICAL JOINTS 21 AND 22)
P_{21-22}
Q_{22-23}
LETTER FOR THE IDENTIFICATION OF EACH HORIZONTAL ROW OF SLABS.
Q-1st ROW OF PERIMETRAL SLABS (HINGE SLABS), CONNECTING WITH THE CUTOFF WALL.
P-2nd ROW OF PERIMETRAL SLABS.

NOTES:

- CONSTRUCTION AND LOADING IN STAGES, WITH SOME IMPORTANT INTERVALS BETWEEN MEASUREMENTS ON ADJACENT POINTS, CAUSED DISCONTINUITY IN SETTLEMENT LINES OF UPPER PART OF DAM.
- THE SETTLEMENTS SHOWN ARE TAKEN FROM THE MEASUREMENTS OF THE UPPER LEFT CORNER SETTLEMENT POINT OF THE SLABS.

FIG. 20.—PLAN OF THE UPSTREAM CONCRETE FACE SHOWING SETTLEMENT

Movements of the vertical joints were already noticed before the first total filling. They involved closing movements in the central part of the dam and opening movements in the lateral areas. The closing movements were more evident at some points in the lower area, in which the concrete face had been completed for some time. At the maximum they reached 2.8 cm (0.09 ft).

The openings showed greater intensity on the right side, between vertical joints 23 and 29, where the greatest were of the order of 2.5 cm. The openings were more moderate on the left side, showing a maximum opening of 1.0 cm (0.03 ft) in the area between joints 0 and 6.

A considerable increase in the movements was seen after the first filling generally in the same direction as had been seen before. Thus, the closing in the central part increased, raising the maximum to 5.6 cm (0.18 ft). In addition, the maximum opening on the right side increased to about 9.8 cm (0.32 ft) at joint 26, and on the left side joints 4 to 6 were slightly closed while the joints to the left of joint 4 had opened slightly to a maximum of 1.0 cm (0.03 ft) (Fig. 22(b)).

In the perimetral joints the most important movements, either for their numerical value or for their repercussions on the water-tightening system of the joints, are the relative movements of the slabs along the common joint. These are identified as sliding movements.

The movements in these joints were seen to be very slight before the first filling. After the first filling, with the exception of the movements of these joints between the vertical joints 12 to 20 whose values are of no significance in virtue of the raising of the joint P-Q, varying movements were noticed that were seen to be much more important than in the previous period.

The maximum values of the accumulated displacements from the beginning of the measurements were: on the left side—opening 3.0 cm (0.10 ft) and closing 1.8 cm (0.06 ft); on the right side—opening 4.5 cm (0.15 ft) and closing 3.2 cm (0.10 ft); sliding (between joints 22 and 23)—10 cm (0.33 ft) (Fig. 23(b)).

Before the first filling only very small movements of the connecting joints were noticed which do not deserve special mention. After the first filling, very small movements on the left side were seen and more considerable ones on the right side. The maximum accumulated displacements of opening and closing at this time were respectively 1.7 cm (0.06 ft) and 3.9 cm (0.13 ft) (Fig. 23(a)).

After the damages to the curtain were verified on the emptying of the reservoir in July, 1958, some repairs were carried out which included chiefly the following (Fig. 24):

1) Replacing the Guttaterna in some perimetral joints where the sticking was defective and was not able to bear the tension to which it was subject by the deformation of the joint, as well as in joint sections where the copper sheet and the Guttaterna yielded due to local deficiencies in the pouring.

2) The grouting without pressure, of cement and sand mortar in the space between the curtain and the placed rock beneath the uplifted perimetral slabs between joints 12 and 20. This was done through holes bored close together and at elevations successively higher, each hole being used only after having allowed the grouted mortar in the previous hole to set. These precautions were taken to prevent the increasing pressure from the mortar lifting the slabs even more and from being lost through the spaces in the placed rock. In this operation 495 cu m (647 cu yd) of sand were used and about 250 tons of cement.

3) The re-opening of some vertical joints to prevent the slabs from touching or becoming strained (Fig. 24). This was done by removing the concrete from one side of the joint making necessary, in consequence, the substitution of part

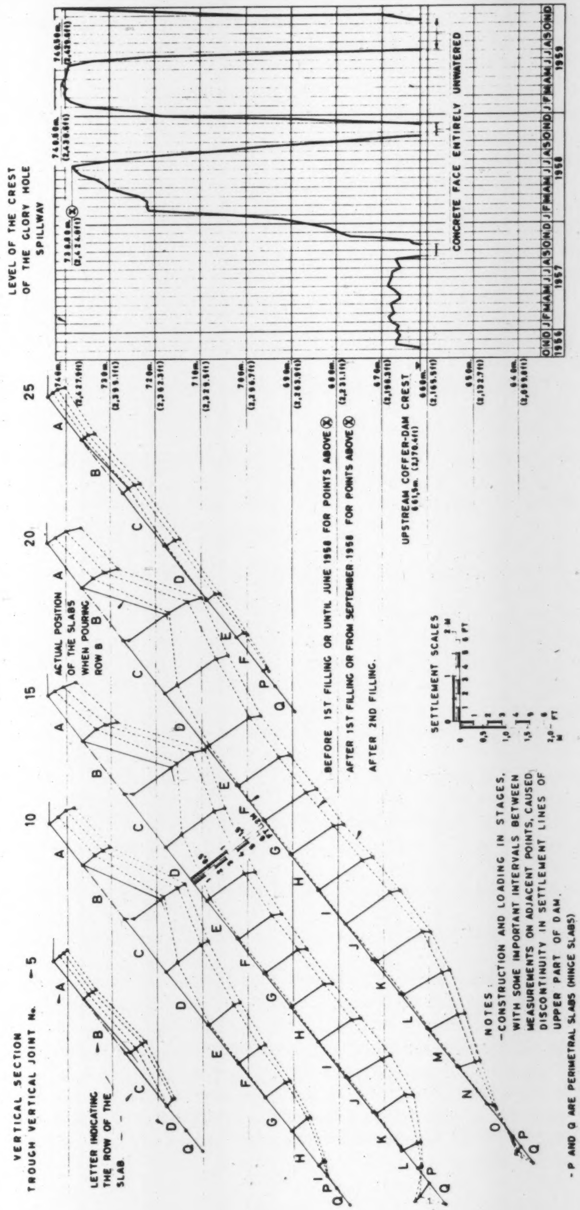


FIG. 21.—SECTION OF THE DAM SHOWING SETTLEMENT

of the copper sheet (the fold and one of the embedded parts) and the relaying of concrete at the edge of the slab leaving a further opening in the joint. In this repair 40 cu m (52 cu yd) of concrete were used.

4) Complete replacing of the fold in the copper sheet of joint 26 that had opened a great deal and was found to be wrinkled by the relative movement of the slabs in the direction of the joint.

5) Repairs to a few local defects in pouring, especially near the copper sheets, and the replacing of some steps in the horizontal joints broken by stones that rolled down during the work on the rockfill of the upper part of the dam.

6) Filling cracks in slabs N 19-20, P 16-17, P 18-19, and P 19-20 with cement grout and superficial reclosing of the same cracks with Guttaterna.

The total cost of these repairs was 600,000$00 Escudos ($21,000).

Observations and Work Carried Out in 1959.—From the date when the elevation of approximately 720 m (2,362.3 ft) was reached, it was seen that the leakage began to be considerably greater than that registered in the previous year for the same levels of water in the reservoir. For example, for the elevation 738.80 m (2,424.0 ft), the maximum reached in the previous filling, the leakage was 1,245 litres per sec (44.0 cfs) as against 382 litres per sec (13.5 cfs).

Until the beginning of the emptying in July, 1959, the water level varied very little, with the maximum of 740.80 m (2,430.6 ft) being reached on April 15, 1959. The leakage on this date was 2,740 litres per sec (96.7 cfs) which increased until June 3, 1959 when the maximum leakage registered was 3.020 litres per sec (106.6 cfs).

It can be seen from Fig. 20 that in consequence of the second filling of the reservoir, there were still considerable displacements in plan, although rather smaller than those measured after the first filling.

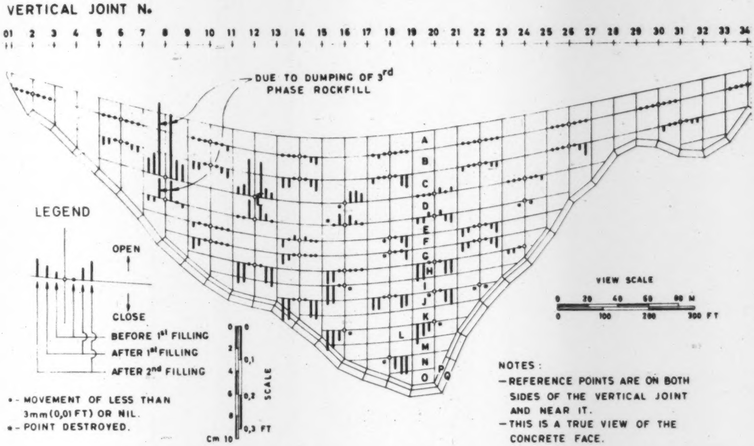
The general movement of the right part of the concrete face towards the left bank is particularly noticeable, so such an extent that at many points the right-to-left movement is seen to be of greater magnitude than the upstream-downstream movement. However, in the left part of the concrete face (from joint 15) the predominance of downstream movements is noted, and the lateral movements lead to settlements in a direction approaching the radial at each point. In addition, one's attention is called to the fact that in the second filling some of these points showed a tendency to settle towards the left.

In the area of the perimetral slabs, between joints 12 and 20, the upstream movement was emphasized, as referred to previously, as a result of the raising of the joint between slabs P and Q.

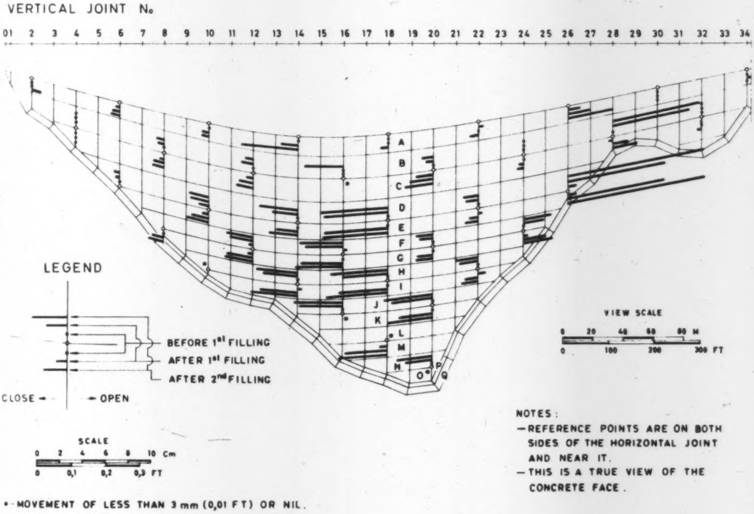
At the end of the second filling in 1959, the maximum absolute displacement downstream was of the order of 1.075 m (3.53 ft) and the maximum lateral displacements from the left to right was 0.169 m (0.55 ft) and from right to left 0.354 m (1.16 ft).

After the second filling, increases in the displacements in profile were observed, as was expected, but they were rather smaller than those registered up to the end of the first filling (Fig. 21). There is, however, an obvious predominance of the vertical component of the settlements, chiefly in the area to the right side of joint 15, in the increase of the displacements of many points.

In accordance with the movements in plan, the profiles show an increase in the raising of the joint of slabs P and Q, already noticed in that area in 1958, between profiles 12 and 20 which emphasizes the space between the concrete face and the placed rock.

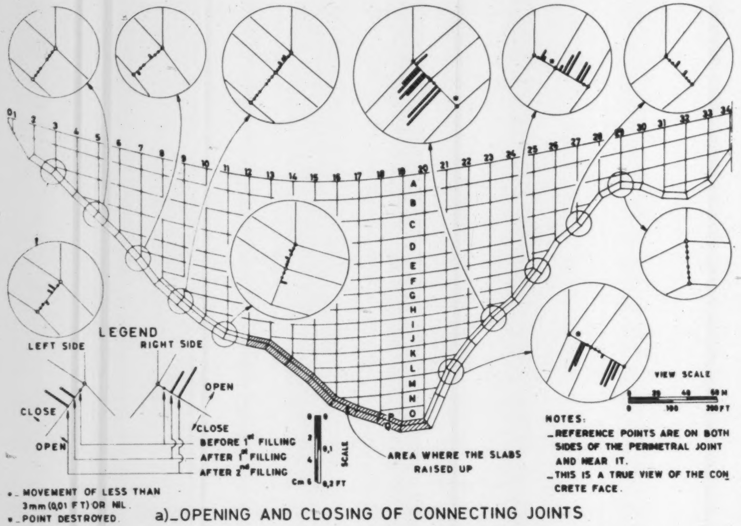

VERTICAL JOINT No
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
DUE TO DUMPING OF 3rd PHASE ROCKFILL
A B C D E F G H I J K L M N O P Q
LEGEND
OPEN
CLOSE
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
• - MOVEMENT OF LESS THAN 3mm (0,01 FT) OR NIL.
• - POINT DESTROYED.
SCALE
0 2 4 6 8 Cm 10
0 0,1 0,2 0,3 FT
VIEW SCALE
0 20 40 60 80 M
0 100 200 300 FT
NOTES:
- REFERENCE POINTS ARE ON BOTH SIDES OF THE VERTICAL JOINT AND NEAR IT.
- THIS IS A TRUE VIEW OF THE CONCRETE FACE.

a)-OPENING AND CLOSING OF HORIZONTAL JOINTS

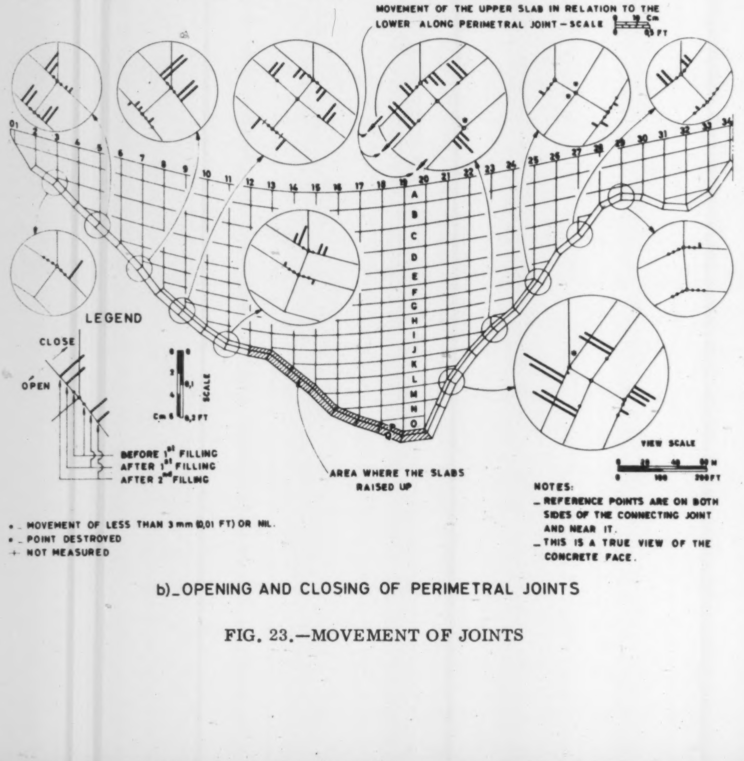

VERTICAL JOINT No
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
A B C D E F G H I J K L M N O P Q
LEGEND
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
CLOSE
OPEN
SCALE
0 2 4 6 8 10 Cm
0 0,1 0,2 0,3 FT
VIEW SCALE
0 20 40 60 80 M
0 100 200 300 FT
NOTES:
- REFERENCE POINTS ARE ON BOTH SIDES OF THE HORIZONTAL JOINT AND NEAR IT.
- THIS IS A TRUE VIEW OF THE CONCRETE FACE.
• - MOVEMENT OF LESS THAN 3 mm (0,01 FT) OR NIL.
• - POINT DESTROYED.

b)-OPENING AND CLOSING OF VERTICAL JOINTS

FIG. 22.—MOVEMENT OF JOINTS


LEGEND
LEFT SIDE
RIGHT SIDE
CLOSE
OPEN
OPEN
CLOSE
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
SCALE
Cm
0,1
0,2 FT
AREA WHERE THE SLABS RAISED UP
VIEW SCALE
100
200 FT
NOTES:
_REFERENCE POINTS ARE ON BOTH SIDES OF THE PERIMETRAL JOINT AND NEAR IT.
_THIS IS A TRUE VIEW OF THE CONCRETE FACE.
• _ MOVEMENT OF LESS THAN 3mm (0,01 FT) OR NIL.
• _ POINT DESTROYED.
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O

a)_OPENING AND CLOSING OF CONNECTING JOINTS


MOVEMENT OF THE UPPER SLAB IN RELATION TO THE LOWER ALONG PERIMETRAL JOINT - SCALE
10 Cm
0,3 FT
LEGEND
CLOSE
OPEN
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
SCALE
Cm
0,1
0,2 FT
AREA WHERE THE SLABS RAISED UP
VIEW SCALE
100
200 FT
NOTES:
_ REFERENCE POINTS ARE ON BOTH SIDES OF THE CONNECTING JOINT AND NEAR IT.
_ THIS IS A TRUE VIEW OF THE CONCRETE FACE.
• _ MOVEMENT OF LESS THAN 3 mm (0,01 FT) OR NIL.
• _ POINT DESTROYED
+ NOT MEASURED

b)_OPENING AND CLOSING OF PERIMETRAL JOINTS

FIG. 23.—MOVEMENT OF JOINTS

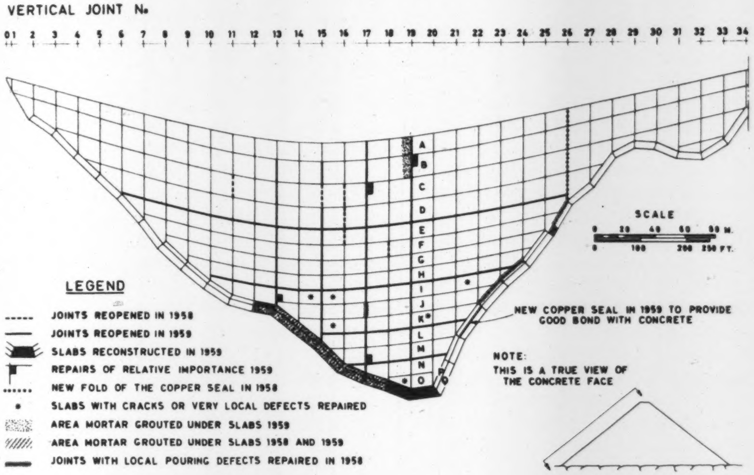

VERTICAL JOINT No
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
A
B
C
D
E
F
G
H
I
J
K
L
M
N
O
P
SCALE
0 20 40 60 80 M.
0 100 200 250 FT.
LEGEND
JOINTS REOPENED IN 1958
JOINTS REOPENED IN 1959
SLABS RECONSTRUCTED IN 1959
REPAIRS OF RELATIVE IMPORTANCE 1959
NEW FOLD OF THE COPPER SEAL IN 1958
SLABS WITH CRACKS OR VERY LOCAL DEFECTS REPAIRED
AREA MORTAR GROUTED UNDER SLABS 1959
AREA MORTAR GROUTED UNDER SLABS 1958 AND 1959
JOINTS WITH LOCAL POURING DEFECTS REPAIRED IN 1958
NEW COPPER SEAL IN 1959 TO PROVIDE GOOD BOND WITH CONCRETE
NOTE:
THIS IS A TRUE VIEW OF THE CONCRETE FACE

FIG. 24.—MAIN REPAIRS AND JOINTS REOPENED

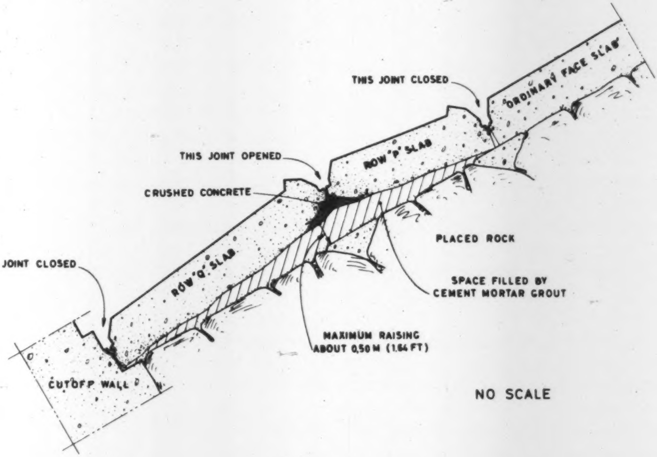

THIS JOINT CLOSED
"ORDINARY FACE SLAB"
THIS JOINT OPENED
ROW "P" SLAB
CRUSHED CONCRETE
PLACED ROCK
THIS JOINT CLOSED
ROW "Q" SLAB
SPACE FILLED BY CEMENT MORTAR GROUT
MAXIMUM RAISING ABOUT 0,50 M (1.64 FT)
CUTOFF WALL
NO SCALE

FIG. 25.—MOVEMENT OF PERIMETRAL SLABS

The total maximum displacement, measured on the plan of the profile and in the direction of the normal to the face, is of the order of 2.04 m (6.69 ft) and the maximum deviation of the displaced point relative to that normal is about 0.39 m (1.28 ft).

The general closing movement of the horizontal joints was still registered after the second filling, although much more moderate and not altering the maximum shown in 1958 (Fig. 22(a)).

The movements of the vertical joints in the same directions as occurred previously continued after the second filling. In the wide area of the dam between vertical joints 4 and 22 there was a closing movement with varying increases whose maximum was of 4.9 cm (0.16 ft). Joints 23 to 29 continued to open in different proportions, with a maximum at joint 26 where there was an increase in opening of 3.8 cm (0.12 ft) thus raising the maximum opening of the joint to 13.2 cm (0.43 ft) (Fig. 22(b)).

After the second filling new movements were seen, in the perimetral joints making the maxima of the accumulated displacements to be: on the left side an opening of 3.4 cm (0.11 ft) and closing 2.4 cm (0.08 ft); on the right side—opening 4.5 cm (0.15 ft) and a closing of 3.2 cm (0.10 ft); sliding (between joints 22 and 23)—12 cm (0.39 ft) (Fig. 4(b)).

After the second filling relatively small movements of the connecting joints were seen.

The maximum accumulated displacements were: on the left side an opening of 0.9 cm (0.03 ft); on the right side an opening of 2.3 cm (0.08 ft) and a closing of 4.5 cm (0.15 ft) (Fig. 23(a)).

In consequence of the relatively high leakages through the dam it was decided to empty the reservoir to inspect the concrete face, an operation which began on June 25, 1959 and ended on August 3, 1959.

Although sound detection equipment was used in the location of the leakages, when the concrete face was under pressure, the clear results that were hoped for were not obtained. It was the visual inspection, carried out by divers working about 10 m (33 ft) down, and completed by observation of the unwatered face that allowed the most important leakage points to be defined.

The most important damage was found to be in slab P 19-20 which showed a crack in the concrete along joint P-Q, that was about 3 m (10 ft) in length and openings on the surface varying between 5 cm and 8 cm (0.16 and 0.26 ft).

Besides this damage, to which the leakage shown can largely be attributed, more was found and can be described in brief as follows (Fig. 24):

1) A large number of the slabs showed almost capillary cracks, the majority of which were only superficial and did not penetrate the full thickness of the slabs.

2) On 4 slabs, B 19-20, C 17-18, I 13-14, and M 17-18, spalling of the layer of concrete above the plane of the reinforcement was observed, localized near the edges of the slabs and only affecting small areas. These damages were caused by the compressions that caused the closing of the joints.

3) On the slab Q 25-26 a crack about 2 cm (0.07 ft) wide on the surface completely cut off one of the corners of the slab.

4) In various stretches of the perimetral joints the Guttaterna was found to be detached from the concrete. This is explained by the opening movements of the joints and by the relative movements of the slabs along the perimetral joints.

5) The perimetral slabs between joints 18 and 20 were very fractured with indications of crushing caused by the compression of the upper slabs against the cut-off wall. It was in slab P 19-20 that the greatest leakage was discovered, as referred to previously.

6) The perimetral slabs between joints 12 and 18 showed an increase in the uplift already noticed the year before (1958) along joint P-Q, as they had become detached from the cement grouted in that year. This uplift was caused by the compression exerted by the slabs adjoining the perimetral slabs (Fig. 25).

7) The perimetral joint common to slabs P and Q 21-22 was misshapen in certain stretches as a result of the yielding of the copper seal due to local deficiencies in pouring around this seal.

8) The perimetral slab P 12-13, the appearance of which was normal except for the raising referred to previously, was broken when the reservoir was emptied about four days after it became unwatered. This occurrence is explained by the compressions already mentioned that increased with the expansion of the slabs under the influence of solar rays.

Most of the damages found were the result of compressions developed in the plane of the slabs because of the general deformation of the rockfill and the closing of the joints, except for some damages that can be attributed to very localized construction deficiencies.

After analyzing the general deformations and the movement of the joints on the concrete-face, and the damages discovered, it was decided to allow new clearance in the joints that were found to be closed or almost closed, with a view to removing or relieving the compression between the slabs and to give greater possibility of settlement without breaking the slabs.

Because of the short time available between the completion of the emptying of the reservoir and the start of refilling, it was impossible to give more clearance to all the joints that had no clearance whatever, or very little, and a careful choice was made of those joints where widening would be more convenient. Thus, the vertical joints 13, 15, 17, and 19 were widened, except for some small stretches that maintained considerable clearance or had been widened the previous year.

The concrete on one side of the joint to be widened was destroyed and a new copper sheet was welded to the part embedded in the remaining side of the joint. The reinforcing that was exposed on the destroyed side was rebuilt, and repouring was accomplished. The total length of vertical joints that were widened was 674 m (2,211 ft). The experience gained in a similar operation the previous year (1958) showed that this repair is entirely efficient.

With regard to the horizontal joints, which from joint D-E downwards were almost all under pressure, it was decided to widen the joints D-E, H-I, K-L, and M-N, which gave a total length of 418 m (1,371 ft) of widened joints. A clearance of 5 cm (0.16 ft) was left in the joints. To fill in the joint, steam-packed cork board was used instead of wood.

Due to the compression between the slabs, special care was taken in beginning the widening process of the horizontal joints, because of the danger of important downward movements in the slabs immediately above the widened joint. This, in fact, did not happen because the movements as seen were normal. In any case, and for the sake of caution, the joints were successively widened from the top downwards.

The operation of widening the horizontal joints was carried out in a manner similar to that in the vertical joints. All the copper seals crossing-pieces were retained, because they were all found to be in good condition.

In filling the cuts that were made in the process of widening the vertical and horizontal joints, 312 cu m (408 cu yd) of concrete were used. At the same time as the work of widening the joints was in progress, repairs to the damage found in the concrete face were carried out. These were as follows (Fig. 24).

1) Narrow cracks in some slabs were reclosed where there were signs of leakage.

2) In the areas of slabs B 19-20, C 17-18, I 13-14, and M 17-18, that showed spalling of the concrete, the concrete was taken off locally until the upper mesh of the reinforcing was exposed and new concrete was poured.

3) In the corner of slab Q 25-26, detached by a crack, the concrete was removed and new concrete was poured.

4) The Guttaterna that had become detached in the perimetral joints was taken out and replaced in position.

5) The perimetral slabs between joints 18 and 20 and the perimetral slab P 12-13 were completely destroyed and new ones were made. In the pouring of these slabs 340 cu m (445 cu yd) of concrete was used.

6) In the perimetral slabs between joints 12 and 18, the underlying spaces were filled, grouting with cement and sand mortar without using pressure.

7) In the perimetral joint common to the slabs P and Q 21-22, the concrete at the edges of the joints was removed and a new copper sheet was placed in position and new concrete poured.

The removal of all of the concrete for the widening of the joints, or for the repair of damages, was carried out without the use of explosives.

Conclusions.—The observation of the upstream face of the Paradela dam, after the second filling and the complete emptying of the reservoir, shows that the concrete face of the dam, accompanying the movements of the rockfill, settled with movements that can be classified as: a) movements normal to the face of the dam; and b) movements tangential to the face of the dam.

Judging from present observations, the concrete-face showed completely satisfactory behavior relative to the movements normal to the face. With regard to the tangential movements, the behavior of the concrete-face was not entirely satisfactory however, as can be seen from the damages occurred in spite of the steps taken in the design with the intention of giving the concrete-face greater possibility to withstand these movements. These steps are seen to be completely justified and without them important damages would have occurred.

The tangential settlements of the concrete face at each point observed can be resolved into two components: one component occurs in the direction of the vertical joints and always downwards. The other component is in the direction normal to the other and directed from the right side towards the center, approximately in the two thirds—central and right—of the upstream face of the dam, and from the left side towards the center approximately in the left third.

The component of the settlement along the vertical joints tends to close the horizontal joints. This settlement is a consequence of the settlement of the rockfill in its direction because of the crushing of the contacts of the rocks, the breaking of the rocks, and the relative slidings of the rocks leading to a more stable arrangement. In dams constructed on a very deformable foundation, the difference of the settlements between the ground and the cut-off wall also helps to increase the component in question. Consequently, excepting the irregularities, the closing of a horizontal joint is greater when the height of the dam above the joint in consideration is greater, the rockfill material is less resistant, the construction of the rockfill in general and especially the placed rock is less efficient, and the foundation of the dam is more deformable.

At Paradela, where the horizontal joints were constructed with an opening of 3 cm (0.1 ft), filled with wood, to a depth of about 50 m (164.05 ft) no horizontal joint closed completely. Beyond this depth the general tendency was for the horizontal joints to close completely in the proportion allowed by the crushing of the wood between them. Once the joints were closed there was an accumulation of the forces on the lower slabs that caused the raising of the perimetral slabs between joints 12 and 20, as has been described.

As an important factor for future designs, it is pointed out that the total settlement of slab O between joints 19 and 20, and in the direction of these, reached 0.26 m (0.85 ft).

With regard to the component of the tangential settlements normal to the direction of the vertical joints it is seen that its direction is predominantly from right to left. This can be explained by the greater slope of the right side in relation to the left side, by the convex shape of the surface on which the rockfill lays on the right abutment, and by the greater deformability of the foundation of the left abutment, formed of heavily weathered granite.

Besides these causes, there may exist another much more important cause, namely the anisotropy of the rockfill resulting from the manner in which it was being dumped. But the writers haven't yet any firm ideas on this matter.

The opening and closing movements of the vertical joints, some even closing completely in certain stretches, are due to the component of the tangential settlements in question and to the variation of the horizontal length of the concrete-face because of the variation in the curvature.

Because the total of the tangential settlements has a direction more or less parallel to the perimetral joints on the right side and approximately normal to the perimetral joints on the left side, small opening and closing movements in those joints and important sliding movements in its direction were observed, while in the perimetral joints of the left side closing movements predominated.

Special attention is called to the important sliding movements seen along the perimetral joints of the right side. For example, in the perimetral joints situated between vertical joints 22 and 23 the total sliding movement in relation to the cut-off wall was 0.29 m (0.95 ft), distributed as follows: 0.10 m (0.33 ft) in the first perimetral joint; 0.12 m (0.39 ft) in the second; and 0.07 m (0.23 ft) in the joint with the cut-off wall. If it had not been for the existence of three perimetral joints and the precautions taken to guarantee the mobility and the water-tightness of the joints, serious damages could have taken place.

The experience gained from the performance of Paradela Dam until now can be summarized as follows:

1) The general distribution of the joints proved to be perfectly efficient.

2) The water-tightening joint arrangements functioned successfully. In spite of this, an attempt should be made to improve, even further, the arrangements of the perimetral joints in order to give better adaptability to the sliding movements along the joints.

3) The vertical joints in the central area of the dam should have greater initial opening to allow a greater closing movement. The vertical joints in the upper part of the right side should have been designed essentially for opening movements.

4) The horizontal joints should have been designed with increasing openings from joint F-G to a maximum of 0.07 or 0.08 m (0.23 or 0.26 ft) in the lower part of the dam and, in place of wood, a more compressible material, steam-packed cork board for example, should have been used. The same applies to the corresponding perimetral joints of the left side.

5) It is very important for the design of the joints to foresee the tangential settlements of the concrete-face, taking into account the slope, the configuration

and deformability of the banks, the rockfill material characteristics and the anisotropy of the loose rockfill according to the carrying out of the dumping.

The study of such anisotropy and the making of tests on the rock utilized in large dams constructed by similar processes is of great interest in order to get information on the influence of the rock characteristics on the yielding of rockfills, and, thus, to be able to foresee more certainly the settlements of the concrete face.

6) The slope 1.3-on-1 of the upstream face, the relatively large thickness of the placed rock wall, and the very careful laying of this wall were factors that contributed greatly to the successful behavior of Paradela Dam. In fact, if the upstream face had been given greater slope and a thinner placed rock wall, or one less carefully layed, the tangential settlements of the concrete face would have been greater, especially in the direction of the vertical joints, either because the stresses in the wall would be greater or because its deformability would have increased. Under these conditions the movements of the joints would have been greater and the resulting damages would have been much more important.

7) The adoption of a relatively important curvature at Paradela has advantages and disadvantages. It is possible to eliminate the disadvantages.

In effect, the existence of the curvature causes the formation of transversal arches inside the rockfill whose deformation tends to intensify the lateral contacts between rocks along the development line of the arches. The intensification of these contacts, thus increasing the lateral compression, makes the settlements along the line of the vertical joints, as well as the slidings along the perimetral joints, a little more difficult. These are the desired advantages and it is believed that they have been obtained although they cannot be appreciated quantitatively.

On the other hand the increase of lateral compression resulting from the shortening of the arches facilitates the closing of the vertical and perimetral joints when the latter tend to close. This occurs with the perimetral joints in the left side of Paradela dam. These are the disadvantages of the curvature, disadvantages that can be controlled by giving those joints greater possibilities of closing. The expression "arch action" is purely comparative but it is used to give an approximate idea of what really takes place.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3077

ROCKFILL DAMS: FOUNDATION TREATMENT OF PARADELA DAM

By Walter J. Weyermann,[1] F. ASCE

With Discussion by Messrs. Judson P. Elston; F. L. Lawton; and Walter J. Weyermann

SYNOPSIS

Foundation conditions at the site of the Paradela Dam in Portugal and the various problems they presented are described. The paper discusses the arrangement of grout curtains and other treatment carried out to suit the geological conditions, and presents performance data on seepage control.

The site of the Paradela Rockfill Dam presented various foundation problems that were resolved by an arrangement of grout curtains and other treatment to suit the geological conditions. The Paradela Concrete Face Rockfill Dam has been completely described by Messrs. Fernandes, de Oliveria and Porto.[2]

Foundation conditions were explored in 19 test drillings totaling 668.54 meters as well as by means of 21 adits the total length of which amounts to 841.20 meters. Most of the drillings are inclined as site surface investigations disclosed that rock joints and faults are approximately in the vertical.

The rock encountered at Paradela Dam is a granite showing decomposition of hypogean character. Owing to marked weathering, one of the exploratory tunnels did not find reasonably compact rock even at a depth of 50 m.

Systematic water pressure tests were carried out in all boreholes, in stages 5 m in length, at two or three pressures, i.e. 2, 5, and 10 kg per sq. cm. on the average. By comparing the rate of water absorption at different pressures, it could be ascertained whether the water flow in the subsoil is turbulent or laminar. In the first case, water absorption is proportional to the square root of the hydraulic head, whereas in the second case the water flow, in accordance with Darcy's formula, is in direct relationship with the

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Note.—Published essentially as printed here in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1748. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Civ. Engr., Swissboring Overseas Corp., Ltd., Zurich, Switzerland, Formerly, Mgr., Sondagens Rodio Ltd., Drilling and Grouting Contractors, Lisbon, Portugal.

[2] "Rockfill Dams: Paradela Concrete Face Dam," by Luis Henrique Gomes Fernandes, Edgard de Oliveira, and Nuno de Vasconcelas Porto, Transactions, ASCE, Vol. 125, Part II, 1960.

head. Tests results fully confirmed the general indications obtained from the cores: the permeability of this highly weathered granite is in the main similar to that of sandy material.

The granite at the dam site is intersected by several rather large fissures or faults. At the time when the corresponding tectonic movement took place, the granite along the faults was crushed into a pulp rich in clay and sand. Further decomposition and weathering utterly transformed this crushed material so that these faults offer now a risk of leakage.

The exact location of the dam was decided upon so as to make it possible to establish the cutoff in a direction coincident with that of the best granite zone. It was, of course, impossible to avoid the faults and seams orientated from upstream towards downstream. In all points were these fissures cross the cutoff, excavating was continued either to a depth where relatively less disintegrated material was found or to a maximum depth of 15 m, from the floor of the inspection gallery within the cutoff wall. At the same time, faults showing insufficient compacity were excavated downstream from the cutoff wall; the extent of this operation is shown in Fig. 2.

A rock-fill dam offers two peculiar features as far as sealing of the permeable foundation soil below the cutoff wall is concerned:

(1) The full hydraulic head from the reservoir at capacity must be met by a very small zone of soil or rock around the contact surface of the cutoff wall.

(2) It is difficult to detect and to stop grout leakage at the base of the rock-fill.

Owing to these two peculiar conditions, it was necessary to supplement the main grout curtain by several means. Two short auxiliary curtains were therefore established upstream and downstream (see Fig. 1), thus providing more extensive direct grouting of the rock in contact with the cutoff wall. Since these curtains were executed first, they created better conditions for grouting the main curtain by reducing the likelihood of grout leakage. Furthermore, the base of the placed rock, between the rock-fill and the concrete facing of the dam, was laid in cement mortar, thus preventing grout leaking in the close neighbourhood of the cutoff.

Timing of all grouting operations was scheduled in accordance with the progress of the dam structure, as deformations of the foundation under the weight of the rock-fill was to be accounted with. Grouting work began at the upstream auxiliary curtain; first, only at points where the rock-fill had

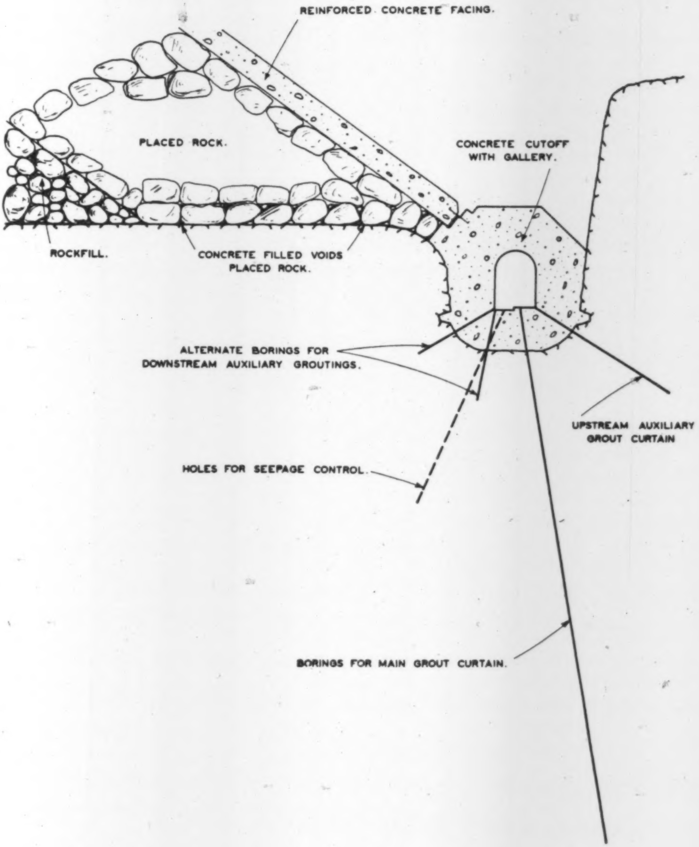

REINFORCED CONCRETE FACING.
PLACED ROCK.
CONCRETE CUTOFF WITH GALLERY.
ROCKFILL.
CONCRETE FILLED VOIDS PLACED ROCK.
ALTERNATE BORINGS FOR DOWNSTREAM AUXILIARY GROUTINGS.
UPSTREAM AUXILIARY GROUT CURTAIN
HOLES FOR SEEPAGE CONTROL.
BORINGS FOR MAIN GROUT CURTAIN.

FIG. 1.—GROUT CURTAINS

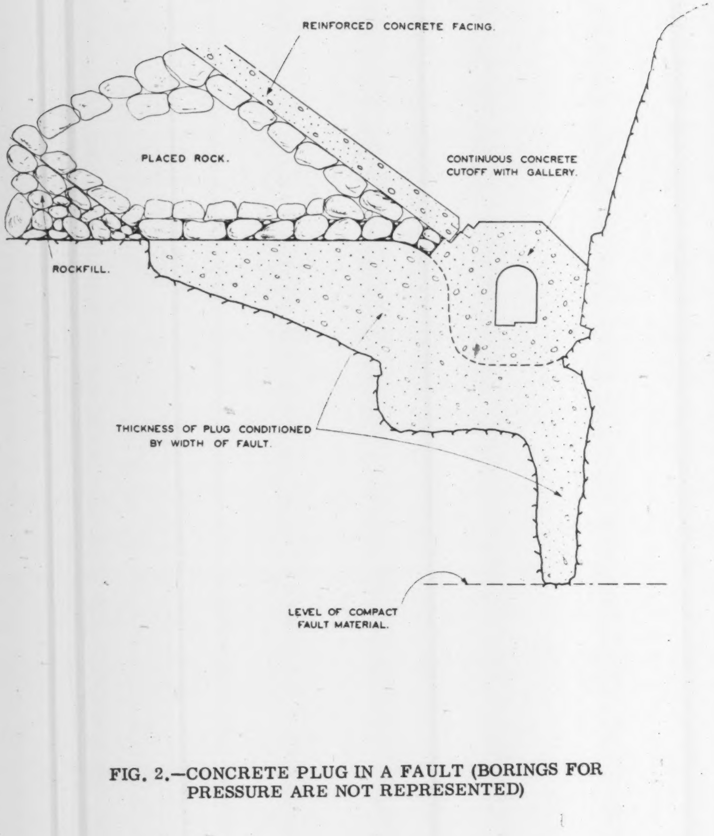

REINFORCED CONCRETE FACING.
PLACED ROCK.
CONTINUOUS CONCRETE CUTOFF WITH GALLERY.
ROCKFILL.
THICKNESS OF PLUG CONDITIONED BY WIDTH OF FAULT.
LEVEL OF COMPACT FAULT MATERIAL.

FIG. 2.—CONCRETE PLUG IN A FAULT (BORINGS FOR PRESSURE ARE NOT REPRESENTED)

already reached elevation 700. In doing so, it could be assumed that no further movements would take place in the foundation, with the exception of those resulting from the filling of the reservoir.

Grouting of the downstream auxiliary curtain was second on the time schedule. Once it was completed, the main curtain was grouted and, finally, with water level in the reservoir already rising, the contraction joints between the elements of the cutoff wall were sealed.

When projecting grouting operations, two kinds of possible water losses from the reservoir had to be reckoned with: (i) leakage through rock joints several millimeters wide and more or less filled up with coarse sandy material, (ii) through fissures filled with material like fine sand and through the mass of disintegrated porous granite whose permeability was known. First, it was proposed to seal the large seams with cement grout, as usual, and to remedy afterwards the subsisting permeability of rock exhibiting minor porosity by means of silicate grouting. But, as the entire upper zone of the reservoir is empty during the dry and hot summer season, decomposition of the silicate as a result of its drying was to be feared. Therefore, another procedure was arrived at and given a trial in a large-scale field test: by improving the efficiency of very finely grained cement grout, a three-dimensional network of sealed joints would enclose the permeable masses of porous granite and, as a result, the effect of the subsisting permeability would be granite and, as a result, the effect of the subsisting permeability would be harmless. Moreover, the rate of permeability of weathered granite being rather low, the occurrence of local water flow would be of very small importance.

Experienced grouting engineers know well that a preliminary injection of silicate gel succeeds in lubricating such fissures where cement grout is scarcely admitted when no special procedure is applied. Field tests repeatedly showed that granite zones that would absorb practically no cement grout in a first attempt, could take in a considerable quantity of cement after the boreholes had been previously washed and a small amount of silicate injected. After such a pretreatment, these zones could be immediately regrouted with cement and through the same drillings.

The above procedure was applied for grouting the main curtain: a first range of holes was drilled and grouted, at 5 m spacing, so as to fill up the main fissures. Once the first range was completed, two intermediate holes were drilled and grouted. Knowing that, as a rule, grout shows a tendency to flow up from the borehole towards the surface, the next groups of grout holes were drilled and treated in downhill sequence whenever possible.

Maximum grouting pressures were determined as follows:

- 20 kg/sq. cm. when the packer was located at a distance of /over 10 m from the concrete of the cutoff wall,
- 10 kg/sq. cm. when the packer of the borehole was located within the concrete of the cutoff wall or a distance not exceeding 10 m,
- 5 kg/sq. cm. for sealing the contraction joints in the cutoff wall.

Depth to be reached by the three grout curtains were set as follows:

- Upstream auxiliary curtain: 5 m down into granite;
- Downstream auxiliary curtain: 2 m;
- Main grout curtain: Drillings were carried out down till they met practically impermeable rock, i.e. down to a 5 m stage whose absorption—checked in a 10 minute test after the water flow had become stabilized—

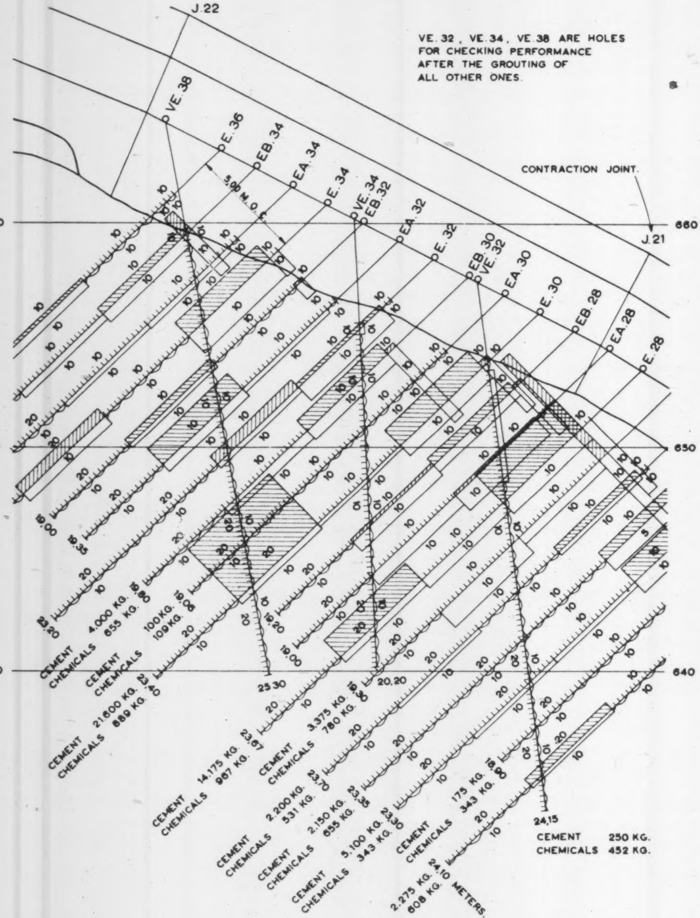

J.22
VE. 32 , VE. 34 , VE. 38 ARE HOLES FOR CHECKING PERFORMANCE AFTER THE GROUTING OF ALL OTHER ONES.
VE. 38
E. 36
EB. 34
EA. 34
E. 34
VE. 34
EB. 32
EA. 32
E. 32
EB. 30
VE. 32
EA. 30
E. 30
EB. 28
EA. 28
E. 28
5.00 M. O. C.
CONTRACTION JOINT.
J. 21
660
650
640
23.20
15.00
CEMENT 4.000 KG.
CHEMICALS 655 KG.
CEMENT 100 KG.
CHEMICALS 109 KG.
CEMENT 21.600 KG.
CHEMICALS 889 KG.
23.40
25.30
15.20
19.00
20.20
23.67
CEMENT 14.175 KG.
CHEMICALS 967 KG.
CEMENT 3.375 KG.
CHEMICALS 780 KG.
23.70
CEMENT 2.200 KG.
CHEMICALS 531 KG.
CEMENT 2.150 KG.
CHEMICALS 655 KG.
23.30
CEMENT 5.100 KG.
CHEMICALS 343 KG.
CEMENT 175 KG.
CHEMICALS 343 KG.
24.15
CEMENT 250 KG.
CHEMICALS 452 KG.
24.10 METERS.
CEMENT 2.275 KG.
CHEMICALS 608 KG.
WATER ABSORPTION BEFORE GROUTING.
10
5
0
8
10
LITERS PER MINUTE AND PER METER.
MEANS THE PRESSURE KG PER CM²
LESS THAN ONE LITER PER MINUTE AND PER METER.
CEMENT ABSORPTIONS.
1.000
500
0
20
KG PER METER.
MEANS THE FINAL PRESSURE KG PER CM²
LESS THAN 100 KG PER METER.

FIG. 3.—FRAGMENT OF MAIN GROUT CURTAIN

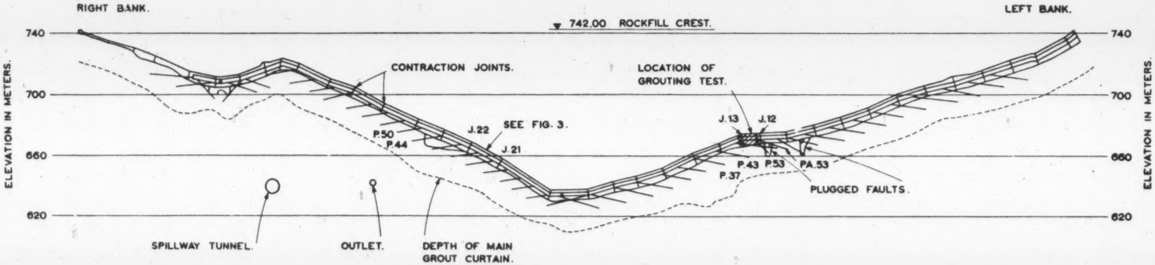
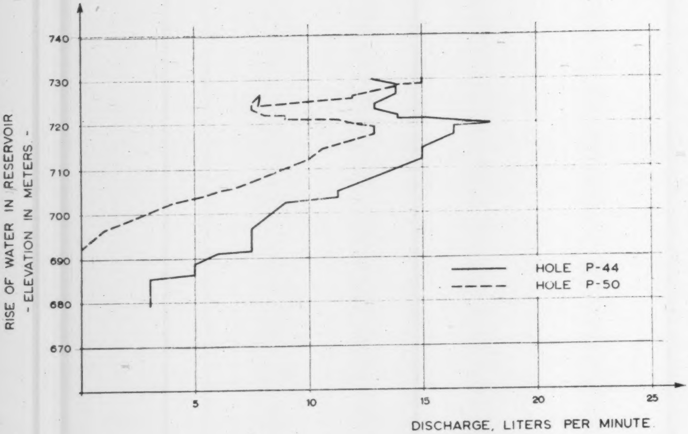

RIGHT BANK.
▼ 742.00 ROCKFILL CREST.
LEFT BANK.
740
700
660
620
ELEVATION IN METERS.
CONTRACTION JOINTS.
LOCATION OF GROUTING TEST.
J.13
J.12
J.22
SEE FIG. 3.
J.21
P.50
P.44
P.43
P.53
PA.53
P.37
PLUGGED FAULTS.
SPILLWAY TUNNEL.
OUTLET.
DEPTH OF MAIN GROUT CURTAIN.

FIG. 4.—DISTRIBUTION OF HOLES FOR SEEPAGE CONTROL (PROFILE ALONG THE CUTOFF)



OF ALL HOLES FOR SEEPAGE CONTROL, P-44 SHOWED THE HIGHEST DISCHARGE.

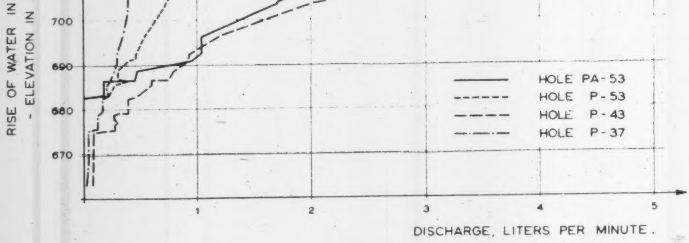


FIG. 5.—DISCHARGE OF SOME HOLES FOR SEEPAGE CONTROL

appeared to be less than one liter per minute and per meter, the pressure applied being 10 kg/sq. cm. A minimum depth of 20 m was ordered.

Amount of grout used, per meter of cutoff wall:

- Cement 1564 kg, chemicals 169 kg, in the three curtains.
- Total drilling: 18.80 m per meter of cutoff wall.

Whenever a water loss from the reservoir is observed in a structure of the type of Paradela Dam, it is impossible to locate the leaks as neither the concrete facing nor the soil area in the close neighbourhood of the cutoff wall are free for inspection. But, considering the danger which the occurence of piping may involve, it is imperative to provide for its immediate location so that measures to prevent a disaster may be taken.

At Paradela Dam this problem was overcome by creating means for a close checking of the behaviour of the grout curtains. An extraordinary large number of seepage control holes were bored after completion of the grouting works (see Figs. 1 and 4). Thanks to these safeguard measures, should piping occur, it would be detected by the unusual inflow into the gallery through the observation borehole controlling the defective zone within the main grout curtain.

The discharges collected in each of these observation holes is recorded by individual graphs. The measurements thus obtained evidence whether the rate of discharge and the head keep a linear character or not. From the readings of most of these graphs, it could be deduced that, as anticipated, the corresponding flow was laminar, i.e. in accordance with the basis of Darcy's formula. Should a curve depart from the linear character as a result of increased discharge, this sign would indicate unsatisfactory conditions and additional grouting would be carried out without delay in the zone controlled by the respective hole. Fortunately nothing of the kind happened.

The graphs furnished also some unexpected and interesting information. Winter rainfall upon the disintegrated granite caused considerable subsurface water flow down the adjoining hills. After a period of several days of dry weather, while the water level in the reservoir was still rising, the discharge from most of the holes for seepage control decreased as indicated in Fig. 5.

The total water losses measured at the downstream heel of the dam amounted to 209.5 litres per second, with the reservoir at elevation 730.16 meters. The total inflow into the cutoff wall gallery, through the holes for seepage control was of 1.4 litres per second at the same time.

DISCUSSION

JUDSON P. ELSTON,[3] F. ASCE.—The paper by Mr. Weyermann was of great interest to the writer. The description of the fault material contained in the granite foundation brings out vividly the tremendous problems that confronted the designers. It would appear that most of the grouting work was conducted in material corresponding to a coarse sand, with generous portions of clay. Mr. Weyermann is to be congratulated upon his excellent presentation of the foundation problems and his detailed thorough description of the steps taken to remedy a very serious situation.

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[3] Asst. Manager Foundations; Uhl, Hall & Rich, Niagara Falls, N. Y.

Extreme difficulty has always been experienced in attempting to grout off silty sands and sandy clays not only with cements but with chemical grouts. It many times has been relatively simple to inject water into such materials but because of solids in the mix larger than the voids to be filled or because of colloidal action, viseosity and specific gravities higher than the water itself, it was found next to impossible to do a satisfactory job of sealing off the materials.

The treatment of the gouge material by using both silica gels and cements appears to be a rather novel and interesting approach. The writer has had considerable experience with cements and with chemicals in foundation treatment. The idea of a combination of grouting materials as described by Mr. Weyermann is certainly a valuable contribution to the science of foundation treatment. It is hoped we shall hear more of the individual foundation problems encountered by engineers throughout the world. There is a growing need for such dissemination of engineering data and practices particularly as we are being faced with poorer foundation materials as time goes on.

F. L. LAWTON,[4] **F. ASCE.**—The author has clearly recognized the two peculiar features associated with the sealing of a permeable foundation below the cut-off wall in a deck-type rockfill dam with concrete face. The grouting scheme adopted for Paradela Dam would appear to adequately deal with these two features, i.e.

1. The full hydraulic head from the reservoir at capacity must be met by a very small zone of soil or rock around the contact surface of the cut-off wall.
2. It is difficult to detect and to stop grout leakage at the base of the rockfill.

It would appear that one very effective means of stopping grout leakage at the base of the rockfill would be the placement of grout covering the desired area with a grout blanket of suitable thickness. This has been done in some cases although perhaps not with deck-type rockfill dams using a concrete facing.

The concrete cut-off with gallery used at Paradela Dam would appear to be a rather expensive method of achieving a satisfactory grout curtain. Would the author care to comment on the relative cost of the grout curtain carried out from the gallery in the concrete cut-off as compared with one carried out in the open?

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[4] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

WALTER J. WEYERMANN,[5] **F. ASCE.**—The author thanks Messrs. Elston and Lawton for their contribution to the discussion of this paper. It was very agreeable to read that the paper on the foundation treatment of the Paradela Dam met with such an interest.

As Mr. Elston points out, dams with simple foundation conditions have already been built, and there remain now to be constructed those with rather complicated conditions. The author thinks this to be the reason why nowadays every dam project requires the collaboration of engineers with special training in foundation treatment.

Mr. Lawton suggests two other solutions which in his opinion might have done for the foundation of Paradela Dam. It would appear, he says, that one very effective means of stopping grout leakage at the base of the rockfill would be the placement of grout covering the desired area with a grout blanket of suitable thickness. But the writer thinks it would have been far more difficult to warrant a reasonable distribution of the grout, as compared with the direct filling of voids with concrete at the base of the placed rock. Especially at the two hillsides a reasonable distribution of grout seems very improbable.

Besides, the concrete filling of voids is cheaper than the pumping-in of grout.

By his second suggestion Mr. Lawton asks the writer to comment on the relative cost of the grout curtain carried out from the gallery in the concrete cut off as compared with one carried out in the open.

The comparison has to be focussed from two view points:

1. The economical; and
2. The technical disadvantages of not having a gallery in the concrete cut off.

From the economical point of view Mr. Lawton is fully right, because a grout curtain carried out in the open would have cost $280,000 - less than the project with a gallery. This figure includes the economy in rock excavation and concrete volume for the cut off wall as well as the absence of formwork for the gallery and the increased expenses of longer grout holes drilled from the open.

But Mr. Lawton himself points out that it is difficult to detect and stop grout leakage at the base of the rockfill. Obviously it is even more difficult to detect and stop water leakage when filling the reservoir.

During the initial storage a water loss of growing importance was measured at the downstream heel of the dam. One can imagine what doubts would have been aroused by a leakage of several hundreds of liters per second if there had not existed those checking means, as pointed out in the paper; a gallery in the cut off, with an extraordinarily large number of seepage control holes. Had there not been this gallery, probably the leakage would have been considered as due to a defective foundation, and logically the reservoir would not have been allowed to reach a high level. If the Hidro-Eléctrica do Cavado had not made full use of the first filling, this company would have had a deficit of placement of approximately 100,000,000 kwh.

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[5] Civ. Engr., Swissboring Overseas Corp., Ltd., Zurich, Switzerland, Formerly, Mgr., Sondagens Rodio Ltd., Drilling and Grouting Contractors, Lisbon, Portugal.

When the reservoir was empty after that first energy producing season, the deductions proved to be right, the reinforced concrete facing of the dam had been broken.

Most interesting were the observations made during the second filling of the Paradela reservoir in spring, 1959. The level went up to El. 741.00 and in spite of the repairs in the reinforced concrete facing, a leakage of about 3,000 liters per sec appeared now at the heel of the rockfill. But thanks again to the gallery in the concrete cut off, it was absolutely certain that the foundation of the dam remained in perfect condition, that is, once more the leakage was due to cracks of the reinforced concrete facing.

The discharges of the seepage control holes were carefully measured and recorded, and the graphs of the second storage proved to be similar to those of the first filling. It could be concluded that there would be no danger with the full charge of the reservoir, and once more the company could increase its output by about 100,000,000 kwh that would have been lost if there had not been a gallery in the cut off.

Modern dams are often projected with impressing height in spite of difficult foundation conditions. Every engineer should feel that such constructions are safe only if there is provision made for an adequate control of their behavior.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3078

ROCKFILL DAMS: MONTGOMERY DAM WITH ASPHALTIC CONCRETE DECK

By F. W. Scheidenhelm,[1] John B. Snethlage,[2] F. ASCE, and
Arthur N. Vanderlip,[3] F. ASCE

With Discussion by Messrs. G. R. Scott; J. L. Sherard; and F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip

SYNOPSIS

This paper describes a rock-fill dam with an asphaltic concrete deck representing a new concept in dam engineering. It sets forth the reasons for selecting that type of dam and deck and explains various aspects of design and of construction, especially as regards the asphaltic concrete deck. It cites the advantages and economy attainable with such type of dam construction.

1. Location and Purpose of Montgomery Reservoir

Montgomery dam is located in Colorado, across the upper portion of the Middle Fork of the South Platte River. The site is 5 miles north of Alma and 11 miles north of Fairplay, Colo. It is nearly 100 miles by highway from Colorado Springs and Denver and about 50 miles from the nearest railroad access at Buena Vista, Colo. The dam is located high on the easterly slopes of the Continental Divide; it is only about a mile from Hoosier Pass across the Divide. (Fig. 1)

The dam and reservoir are owned by the City of Colorado Springs. They form a part of the municipal water supply system. The reservoir has the primary function of seasonally storing water which reaches it through a nearby tunnel under the Continental Divide, having been diverted from headwaters of the Blue River, a tributary of the Colorado River.

The reservoir is expected to be filled each summer as the result of the melting of heavy accumulation of snow on the westerly side of the Divide. The water to which the city is entitled will reach the city from the reservoir by

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Note.—Published essentially as printed here, in February, 1958, in the Journal of the Power Division, as Proceedings Paper 1556. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Cons. Engr., New York, N. Y. (Deceased)

[2] Cons. Engr., East Chatham, N. Y.

[3] Cons. Hydr. Engr., New York, N. Y.

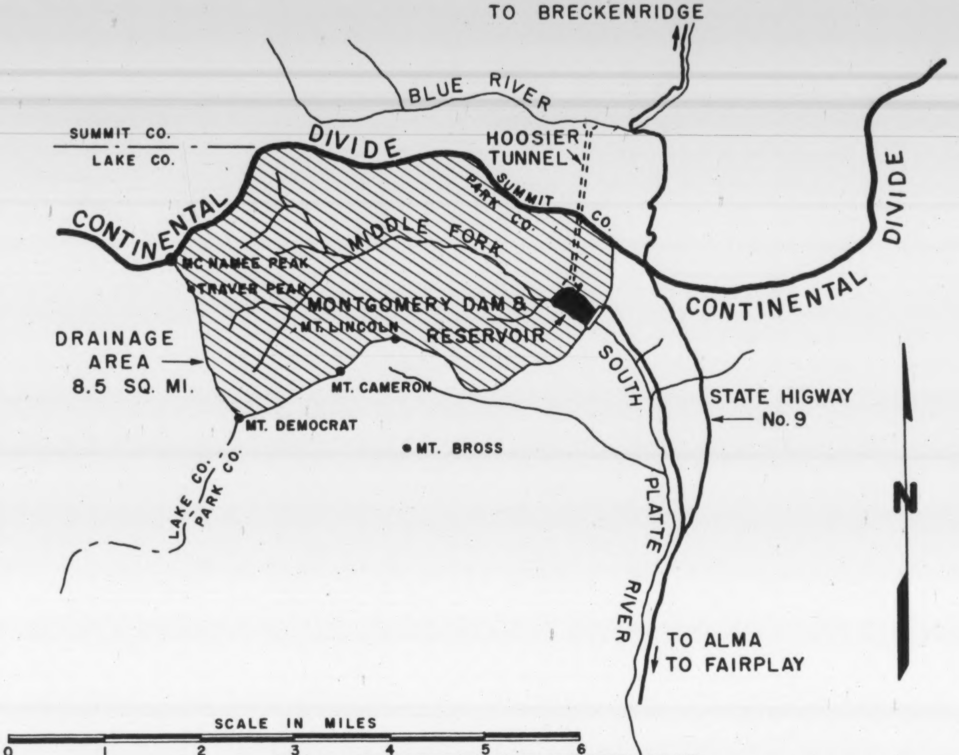
TO BRECKENRIDGE
BLUE RIVER
SUMMIT CO.
LAKE CO.
DIVIDE
HOOSIER TUNNEL
SUMMIT CO.
PARK CO.
CONTINENTAL
CONTINENTAL
DIVIDE
MIDDLE FORK
MC NAMEE PEAK
TRAVER PEAK
MONTGOMERY DAM & RESERVOIR
MT. LINCOLN
DRAINAGE AREA 8.5 SQ. MI.
MT. CAMERON
MT. DEMOCRAT
MT. BROSS
LAKE CO.
PARK CO.
SOUTH PLATTE RIVER
STATE HIGWAY No. 9
N
TO ALMA
TO FAIRPLAY
SCALE IN MILES
0
1
2
3
4
5
6

FIG. 1.—MONTGOMERY DAM, LOCATION PLAN

gravity flow through a 30-in. steel pipeline which feeds into existing reservoirs on the northwest slope of Pike's Peak, thus connecting directly with the city's water supply system.

2. Dam and Reservoir Remarkable in Several Respects

Several exceptional circumstances obtain with regard to the dam and reservoir. Outstanding is the fact that they are located at an altitude more than two miles above sea level. Apparently, in terms of altitude, the structure is the highest dam of substantial size in North America. Incidentally, that altitude has an effect on human performance and otherwise; during construction, the effect was manifest adversely even as regards the efficiency of gasoline engines.

No Middle Fork water may be taken for the use of the city. That is, in the pertinent respect the city's water rights are limited to run-off from the Colorado River drainage area, on the opposite side of the Continental Divide.

The natural drainage area above the dam, being only 8.5 square miles, has no significance except as regards spillway provision at the Montgomery dam. The reservoir is exceptionally small for so long and costly a dam, but such is the price of the much needed water.

Then there is the short annual construction season, of 2-1/2 to 6 months, depending upon the particular operation involved.

Finally, there is the fact that at Montgomery dam there appears to be the first application in the United States of an asphaltic concrete deck to a rock-fill dam. Also it appears to be the first application in the world of this particular type of asphaltic deck to a rock-fill dam.

3. Principal Data and Dimensions (Figs. 2 and 3)

The crest of the dam, exclusive of camber allowance (for settlement) of a maximum of 3 feet, is at elev. 10,868 above Mean Sea Level, U. S. Geological Survey. However, it is contemplated that the height of the dam may in the future be increased, perhaps by as much as 50 feet, and the design, except as to spillway, provides for such raising.

The spillway channel is at the north end of the dam, constructed in part with a lining of reinforced, Portland cement concrete. It has an ungated overflow section of ogee form, with a net width of 97 ft. The crest of the spillway proper, i.e., the ogee, is at elev. 10,861.

The area of the reservoir, when full to spillway crest, is only about 95 acres. The entire reservoir volume, of about 5,100 acre-feet, is utilizable. Should the dam be subsequently raised, as above mentioned, the reservoir capacity would be practically doubled.

The dam is of rock fill, comprised of granite quarried from the mountainside upstream from the right abutment. The watertight diaphragm or deck, resting upon the upstream slope of the dam, is of asphaltic concrete, which was aimed to be constructed in continuous layers and to have the equivalent of a uniform final thickness of 12 inches.

At its maximum section, the dam has a height of 113 ft. above the foundation rock. The length of the rock fill is approximately 1,900 ft. Including the spillway, the length of the dam across the river valley is about 2,000 ft.

The upstream slope of the dam is 1 (vertical) on 1.7 (horizontal); the downstream slope averages 1 on 1.4.

Water from the reservoir is drawn through a screened intake, completely

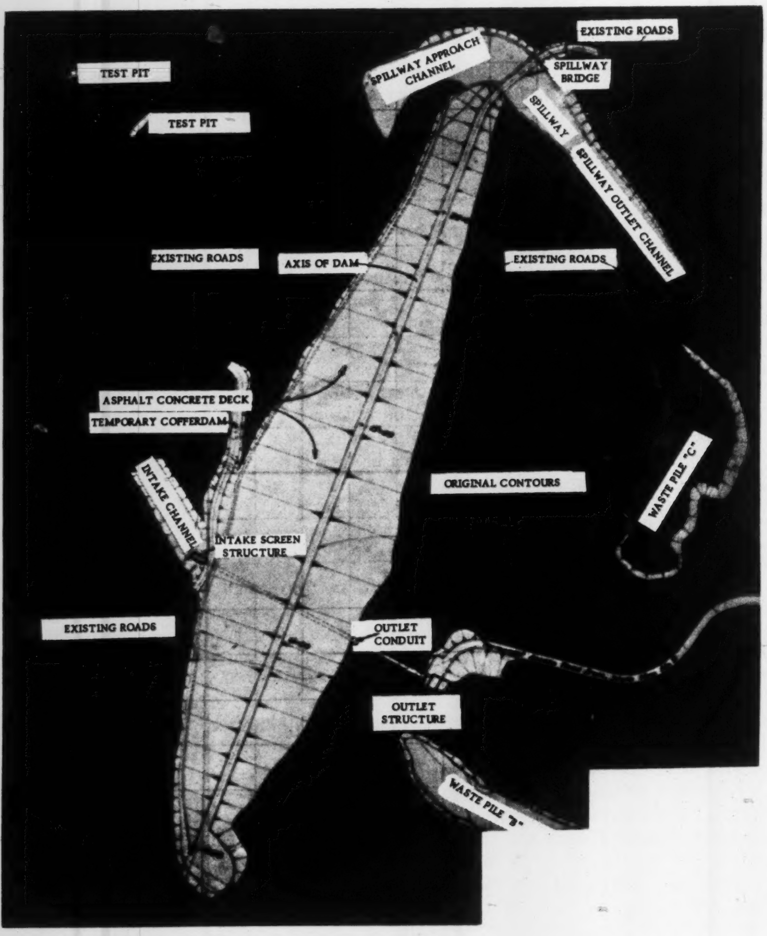
EXISTING ROADS
SPILLWAY APPROACH CHANNEL
SPILLWAY BRIDGE
TEST PIT
TEST PIT
SPILLWAY
SPILLWAY OUTLET CHANNEL
EXISTING ROADS
AXIS OF DAM
EXISTING ROADS
ASPHALT CONCRETE DECK
TEMPORARY COFFERDAM
WASTE PILE "C"
ORIGINAL CONTOURS
INTAKE CHANNEL
INTAKE SCREEN STRUCTURE
EXISTING ROADS
OUTLET CONDUIT
OUTLET STRUCTURE
WASTE PILE "B"

FIG. 2.—MONTGOMERY DAM, PLAN

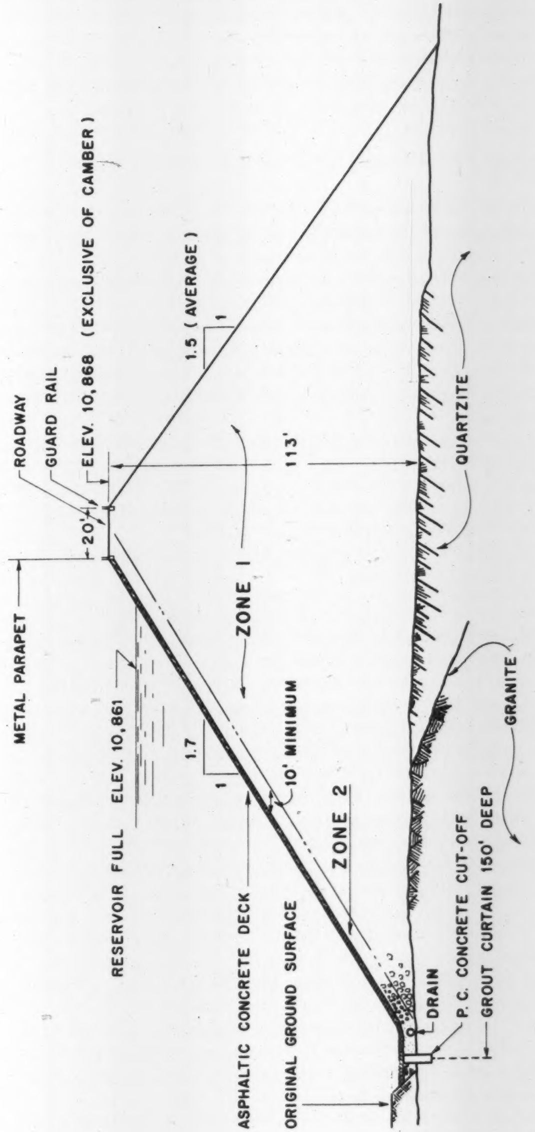
ROADWAY
GUARD RAIL
ELEV. 10,868 (EXCLUSIVE OF CAMBER)
METAL PARAPET
20'
RESERVOIR FULL ELEV. 10,861
1.5 (AVERAGE)
1
113'
ZONE 1
1.7
1
10' MINIMUM
ZONE 2
ASPHALTIC CONCRETE DECK
ORIGINAL GROUND SURFACE
QUARTZITE
GRANITE
DRAIN
P. C. CONCRETE CUT-OFF
GROUT CURTAIN 150' DEEP

FIG. 3.—MAXIMUM CROSS SECTION

submerged except when the reservoir is well-nigh empty. From this intake, a reinforced concrete conduit, of 6 ft. net diameter, 540 ft. long and constructed under the dam in a trench excavated in the foundation rock, conducts the water to an outlet-valve structure. In this structure water can be diverted either into the city's gravity supply line or discharged into the bed of the river. The water which is diverted into the city's pipeline is measured by means of a Parshall flume.

4. Selection of Site

The purpose of the reservoir could have been served by a location almost anywhere downstream from the outlet of the tunnel under the Continental Divide. However, mainly because of the need of head for the gravity-flow pipeline, a location relatively near the tunnel outlet had an important advantage over one any considerable distance downstream.

Serious consideration was given to two sites, one about 3-1/2 miles above the town of Alma and one which would overflow what remained of a ghost mining town called Montgomery. After considerable exploration by drilling, test pits and trenches, together with comparative estimates of cost, the Montgomery site was selected.

The Alma site would have been in sedimentary rock. The Montgomery site is mainly in sedimentary rock, largely quartzite, but roughly one-half of the sealing line at the heel of the dam is in granite, for under the southerly half of the dam the line of contact of the outcrops of the sedimentary rock and the granite lies between the longitudinal axis and the heel.

5. Types of Dam Considered

The width of the valley at the crest of the initial structure being approximately 2,000 ft., an arch dam was out of the question. It was found that any gravity dam of Portland cement concrete, whether of the massive type or of the buttress or so-called hollow type, would be far more costly than a dam of the fill type. Contributing reasons were the long haul for such cement and the short annual construction season, with relatively low temperatures.

Within reasonable proximity to the damsite there is an abundance of glacial deposits, the finer fractions of which might have been barely suitable for rolling into an impervious mass. However, greater estimated cost and the brevity of the season free from freezing temperatures eliminated a dam comprised entirely of rolled earth fill, moistened for the process of rolling. The same factors obtained with regard to the combination of rock fill with an interior, impermeable earth core, whether sloping or central; in addition, the fact that provision had to be made for future raising of the dam in a substantial amount made impracticable the use of any interior earth core except at materially greater initial cost.

Under the circumstances, including the abundance of suitable rock in the abutting mountainsides, it appeared natural and appropriate to select the type wherein the body of the dam is entirely of rock fill and there is an appropriate watertight diaphragm or deck upon the upstream sloping face of the fill. Moreover, with a deck of asphaltic concrete, the estimated cost was less than that of any other conventional type.

In order not to overlook any possibilities, particularly from the standpoint of materials available for the fill, some consideration, though not to the extent of study for design purposes, was given to the use of the unconventional combination of earth only semi-compacted with a like deck of asphaltic concrete. However, in the light of pertinent conditions for Montgomery dam, such a combination was found not to indicate sufficiently lower cost to offset the advantages of the combination of rock fill with asphaltic concrete deck.

6. Advantages of Rock Fill Dam of Deck Type

The more important advantages of rock fill, specifically of the deck type, were considered to comprise the following:

Provided only that there is a watertight diaphragm or deck on the upstream slope, the stability of a dam of dumped rock fill is assured. Uplift is not a factor.

Further, because of the openness and free-draining nature of rock fill and in the absence of an utterly improbable large break extending through the deck, leakage through the dam would do no damage to the fill proper; it would not endanger the stability of the dam.

Correspondingly, grouting would be necessary only to minimize leakage past the dam, not to eliminate uplift—as in the cases of mass gravity or some hollow dams of Portland cement concrete—or to keep leakage water out of the body fill, as would be essential in the case of an earth-fill dam.

Again, because of the foregoing, rock-fill dams of the deck type require a minimum of site exploration.

The deck type of dam would be particularly suited to permit various construction activities to be carried on simultaneously. Thus the construction of a cut-off wall and extending below it a pressure-grouted diaphragm, each using Portland cement, could be carried on at the same time that the rock fill was being placed, although of course not at the same location within the area of the dam. This advantage was of special importance at Montgomery dam because of the short annual construction season. Furthermore, as concerns the body of the dam, the use of rock fill would permit stretching the all-too-short construction season to its utmost.

In the event of overtopping, whether by floods at part height of the dam during construction or by waves after completion, the rock fill would be less subject to damage, if any, than would be true of an earth fill. This factor also had some bearing upon the extent of provision for freeboard (on account of waves) above maximum flood levels.

As compared with a fill dam having an interior core or a sealing diaphragm, the deck-type of fill would more readily lend itself to future increase in height. In the pertinent respect, increase in height would be accomplished by mere extension of the asphaltic concrete deck upward at the same slope and corresponding extension of the cut-off and sealing provisions up the abutments.

Moreover, the sealing into the foundation being at the heel of the dam, such type would afford future access, if and when needed, to the cut-off wall and the appurtenant above-rock sealing provisions, provided only that the reservoir could be drawn down. It happens that in normal operation the Montgomery reservoir will be drawn down annually.

Finally, the estimated cost of a rock-fill dam of the deck type was well within reason. It was less than the estimated cost for any other type of dam affording equivalent advantages.

7. Selection of Material for Deck

In choosing the material of which the watertight diaphragm or deck of the rock-fill dam was to be constructed, there were considered, as offering possibilities of economic feasibility, three kinds of material, to wit: steel, asphaltic concrete and reinforced Portland cement concrete.

Naturally in the selection of the material in point, estimated cost was a primary factor. It was essentially on the ground of higher cost that steel was eliminated.

Basis of Choice of Asphaltic Concrete

Because of its pertinent characteristics, the use of asphaltic concrete as a material for the construction was approved by Black & Veatch, engineers for the City, when proposed by the authors, as consultants to the former. The authors had had prior experience in the use of asphaltic material in dam construction. Support for the use in the present instance of asphaltic concrete was available also in the history of its use in hydraulic construction elsewhere, as is in part set forth under item 8 hereof. For some instances of use of asphaltic concrete in hydraulic structures, the case history and results over a material extent of time were known. And of course there was available a tremendous amount of information with regard to the use of the material in the construction of highways and airfield runways, though only a portion thereof was directly pertinent.

The selection of the material was made with full realization that existing pertinent information would have to be supplemented by extensive testing. Indeed, the selection was tentative in the sense of being subject to favorable outcome of such testing. It was early recognized that, for the purpose in point, the criteria for satisfactory performance would be different from those currently used for asphaltic concrete in highway work and airfields and probably quite different from those so far used for other hydraulic structures.

During the period November 1953-January 1954, there were carried through tests on locally available aggregates (glacial deposits) sufficient to permit tentative design of a stable mix, mainly for the purposes of cost estimating. These tests established beyond doubt that those local aggregates could be appropriately and economically blended so as to produce excellent gradation and that, when combined with a suitable asphalt, they would yield a workable and stable mixture. Final selection of asphaltic concrete was made only after completion of considerable, though not all, of the pertinent testing. Further reference to that testing appears at item 9 hereof.

Comparison with Portland Cement Concrete Deck

The larger rock-fill dams of the deck type heretofore constructed in this country use an impervious deck of Portland cement concrete on the upstream face. A major element of such structures, involving much time and cost, is the construction of a base for the concrete deck in the form of a thick semi-hand-placed or crane-placed layer of relatively large rocks over the mass of dumped rock. Such layer is considered to be primarily a finish and leveling course, containing fewer voids than the underlying rock mass.

The tendency has been to reduce the thickness of such layer of placed rock. But, even so, the practice is to use a minimum thickness of the order of 8 ft., with greater thickness in lower portions of the higher dams. Also, either there must be slots for beams to support the joints of the panels of Portland cement concrete or at least, under the planned positions of the joints, there must, by means of like concrete, be provided flat surfaces upon which to construct the joints and to permit them to function properly.

To be sure, the use of an asphaltic concrete deck involves the preparation of a suitable stone base over the upstream face of the rock fill, but such is relatively simple and less costly to construct. There is eliminated the need for the layer of carefully placed base rock and the beams or other joint supports as normally used in a deck of Portland cement concrete.

For Montgomery dam, comparative estimates of cost as between the two kinds of concrete deck indicated a saving of, at the very least, 50% by the use of asphaltic concrete.

At ordinary temperatures, asphaltic concrete is far more flexible than Portland cement concrete. The laboratory investigations made for the purpose of the dam design and likewise field observations show that asphaltic concrete for the intended purpose may be expected to have considerable flexibility at temperatures practically as low as freezing. The evidence indicates that an asphaltic concrete deck will be self-adjusting to any settlement normally to be expected in a reasonably well constructed rock-fill dam. This does not mean, however, that in construction one may disregard the need for keeping settlement, especially localized settlement, to a minimum.

From the evidence available, there is good reason to believe that asphaltic concrete, when properly designed, placed and compacted, will be at least as durable as competitive materials for such deck construction. An interesting example of the use of either an asphaltic mortar (in that case bitumen and loam) or asphaltic concrete (bitumen, sand and gravel) as used for construction in antiquity and surviving to the present time is given by one Andrae in Mitt. Deutsch. Orient. Ges. XXII, p. 33. The material was used on a mile-long embankment along the banks of the Tigris River about B.C. 1300. Andrae states that "After 3300 years, it still faithfully fulfills its purpose". (Asphalts and Allied Substances, by Herbert Abraham, 5th Ed., 1944, Vol. 1, p. 29.)

In the case of Montgomery dam the type of asphalt, the richness of the mix and the thickness of deck are expected to eliminate certain shortcomings which have been observed in the performance of some asphaltic concrete used in road surfaces not subject to substantial kneading action by traffic.

In this connection there is a further consideration, though one not relied upon in the case of Montgomery dam, namely, that there is some reason for expecting that, if some cracking should occur, the asphaltic concrete may be somewhat self-healing. At any rate, assuming in each instance accessibility for repair, a deck of asphaltic concrete is believed to be more easily repaired than is one of Portland cement concrete.

Of great importance, especially in situations like that at the Montgomery site, is the fact that the covering of a given area of deck surface requires far less time with asphaltic concrete than with Portland cement concrete. This may well have saved a year in the total time of construction of Montgomery dam. As it turned out, the summer of 1957 afforded an even shorter period, during which construction was tolerable, than the usual all-too-short season. In terms of conditions suitable for asphaltic concrete work, the total period available was hardly 3 months, whereas normally it would have been about a month longer. Various delays prevented placing of asphaltic concrete until July 22; yet placement of the 27,770 sq. yds. of the asphaltic concrete deck was completed on September 24—or in about two months.

The nature of asphaltic concrete and the manner of placing and compacting it, even with equipment not in all respects designed for the purpose, are such as to permit very rapid, if not continuous, placing. The objective is to place and finish a given strip of asphaltic concrete before the previously placed, adjoining strip has become cold. Also, if necessary, one layer can be placed upon another with only a few hours' lapse of time.

8. Prior Use of Asphaltic Concrete in Hydraulic Structures

The use of asphaltic concrete in hydraulic structures is of course not new. In various forms and for various purposes it has been used in connection with reservoirs. For instance, in the United States, asphaltic concrete has been used to line water-supply reservoirs of the City of Los Angeles, but not as a watertight membrane. Instead, the asphaltic concrete, in a relatively thin layer designed to be porous throughout, was used on earth-embankment slopes in lieu of riprap, i.e., for slope protection only.

Similar use has been made of asphaltic concrete by the Bureau of Reclamation after experimentation at the site of its Bonny dam in eastern Colorado, namely, in lieu of riprap to protect the upstream slope of its Glen Anne dam in California. (See Civil Engineering, June 1952; Western Construction, September 1953.)

There exists in this country only one example where asphaltic concrete is relied upon to provide watertightness for the slopes of a water-supply reservoir, namely, Turner reservoir constructed in 1952 by the City of Salem, Oregon. (See Western Construction, September, 1952.)

Of some pertinency also is the asphalt-mortar cut-off wall constructed about 1940 at the instance of the authors in the left abutment of Claytor dam on New River, Virginia. There a sheathed trench, approximately 3-1/2 ft. wide and 325 ft. long, was excavated, partly through earth but mainly through badly weathered rock, to a maximum depth of nearly 100 ft. below reservoir level. This was then filled with a mixture of asphalt and rock fines in order to prevent leakage from the reservoir around the left end of the dam proper, the latter being of the Portland cement concrete, gravity type. (See Engineering News-Record, March 13, 1941; The Asphalt Institute Quarterly, April 1951.)

More extensive use of asphaltic concrete to provide a watertight diaphragm for rock-fill dams has been made in foreign countries, though it appears that in no case has such use been the same as that at Montgomery dam.

There follows a listing, with short description, of the principal, if not all, rock-fill dams in the construction of which asphaltic concrete has been or may have been used. In all cases, except the last cited, the material was used to form a watertight diaphragm or deck resting upon the upstream face or slope of the dam:

Bou-Hanifia dam, Algeria, 180 ft. high, completed in 1938.—Deck consists of two layers of porous cement concrete separated by 5 in. of asphaltic concrete, in turn in two layers with wire reinforcement between the latter.

Caritaya dam, Chili, 125 ft. high, completed in 1935.—Deck consists of 3/4 in. of asphalt (perhaps asphaltic mortar) between two layers of reinforced concrete.

Diga di Codelago dam, Italy, 64 ft. high, completed in 1893.—Deck consists of a 2 in. mastic layer, covered by stone blocks set in ordinary cement mortar.

Fedaia dam, Italy, 53 ft. high, completed in 1954.—Deck consists of 4 in. of asphaltic concrete between two layers of ordinary cement concrete. Details not available but the outer layer is reinforced, the inner porous.

Genkel dam, Germany, 140 ft. high, completed in 1952.—Deck consists of two thin layers of asphaltic concrete, separated by a layer of gravel (sparsely bound with asphalt) to provide drainage for such leakage as may occur through the outer layer. The outer of the two layers is 3-1/2 in. thick (placed in 3 sub-layers); the inner a little less than 2-1/2 in. (placed in 2 sub-layers). Under the latter and over the upstream face of the rock fill itself, there is a layer of porous cement concrete.

Ghrib dam, Algeria, 235 ft. high, completed in 1936.—Deck consists of two layers of porous cement concrete separated by 5 in. of asphaltic concrete placed in two layers.

Henne dam, Germany, 200 ft. high, completed in 1954.—Deck is similar to that of Genkel dam.

Iril-Emda dam, Algeria, 246 ft. high, completed in 1954.—Deck consists of 5 in. of asphaltic concrete in two layers, between layers of ordinary cement concrete, the upper of the latter being reinforced, the lower porous.

Mulungushi dam, South Africa, 117 ft. high, completed in 1935.—Is stated in an American textbook to have an asphaltic concrete deck 4 to 12 in. thick. However, the references are vague or incorrect and, although research disclosed various aspects of the water development of which Mulungushi dam is a part, no details could be found either as to the asphaltic concrete deck or as to the pertinent construction methods.

Vrla II dam, Yugoslavia, 82 ft. high, completed in 1955.—Deck includes two thin layers of asphaltic concrete (with wire reinforcement between them) covered by a layer of reinforced concrete.

Vale Do Gaio dam, Portugal, 165 ft. high, completed in 1952.—Here asphaltic concrete is used as part of a sloping interior diaphragm or core, consisting of 4 to 8 in. of asphaltic concrete on an inclined cemented masonry wall, with a slope of 1.4 on 1 and buried under an upstream earth fill.

The foregoing of course does not take into account rock-fill dams where asphalt or asphaltic mortar has been applied as a thin coating over steel, wood, Portland cement concrete, etc., where reliance for watertightness was on other materials or where asphalt was used for preservative purposes rather than as a water barrier.

It is appropriate to mention four foreign dams in which asphaltic concrete was used, not on rock fill but, on earth fill, to greater or less extent for the purpose of ensuring watertightness:

Amecker dam, Germany, 40 ft. high, completed in 1934.—Deck consists of 2 in. of asphaltic concrete, but it is not clear that there is reliance thereon for watertightness.

Oued-Sarno dam, Algeria, 92 ft. high, completed in 1952.—Deck consists of 2-3/4 in. of asphaltic concrete.

Schevelinger dam, Germany, 50 ft. high, completed in 1940.—Deck consists of 2-1/2 in. of asphaltic concrete.

Turawa dam, Germany, 43 ft. high, completed in 1937.—Deck consists of 2 in. of asphaltic concrete.

In view of the foregoing, it appears that the use of asphaltic concrete at Montgomery dam, as the sole material for the deck of a rock-fill dam, is the first instance of such use in the Western Hemisphere and that the methods and details of construction at Montgomery dam are unique and involve the first application thereof anywhere.

9. Experimental Basis for Design and Specifications for Asphaltic Concrete Deck

Review of the more pertinent sets of specifications for asphaltic concrete work made clear that the design and the preparation of specifications for an asphaltic concrete deck for Montgomery rock-fill dam would involve considerations and criteria not heretofore fully taken into account.

Among the more important of these may be mentioned:

Certain aspects of watertightness of the asphaltic concrete.

Behavior of the asphaltic concrete when placed upon relatively steep slopes.

Effect of extremely high temperatures upon flow on given slopes.

Effect of variations in types of aggregate, asphalt binder, etc., upon slope-flow characteristics.

Limiting steepness of slopes.

Effect of freezing and thawing and of wave action upon the asphaltic concrete.

Determination of appropriate thickness of the asphaltic concrete deck.

Placing of such thickness, as concerns number of layers, position and treatment of joints, etc.

Special equipment or modifications of existing equipment to be specified.

Nature and preparation of base.

Manner of making watertight connection of the asphaltic concrete with structural features constructed of Portland cement concrete, such as the cut-off wall and the south wing wall of the spillway.

The solution of most of these problems was concluded to warrant laboratory investigation. For some others, there was resort to trial in the field.

The reason why many of the problems related to asphaltic concrete in such a hydraulic structure had not already been adequately covered is undoubtedly to be found in the differences of conditions and requirements to be met in the several fields of application. Most of the previously available information related primarily to the design and construction of highway and airfield pavements—relatively little to the field of hydraulic structures. A recent compilation and discussion in this field appears in the "Proceedings of the First Western Conference on Asphalt and Hydraulics", published as Bulletin No. 78 by the University of Utah, 1955.

On the other hand, basic research as concerns asphalt, aggregates and asphaltic concrete mixtures had been and of course is being done in those two principal fields of earlier activity. Much fundamental experimentation as to the properties and uses of asphalt has been carried on individually and collectively by producers of petroleum, the results of which experimentation are applicable to the design of asphaltic concrete mixes in any specific field.

For the experimental work required for the construction of Montgomery dam, it was found that nowhere else than at the Engineering Laboratories of the Bureau of Reclamation, Denver, Colorado, was there a laboratory suitably equipped to carry out that experimentation. For this reason and further because of the relative proximity of that laboratory to the site of the dam, arrangements were made with the Bureau for the doing of the work in its Bituminous Laboratory. The arrangement was facilitated, not only because the project in point was that of a municipality of the State of Colorado but also, because the Bureau has before it many like problems involved in the application of asphaltic concrete in the field of hydraulic structures. All of the experimental work in the Bituminous Laboratory was done by laboratory personnel. The Bureau of Reclamation and its personnel were not requested to and did not interpret the test results or directly or indirectly assume any responsibility for interpretation.

The more important categories of laboratory experimentation so undertaken and some of the results were briefly as follows:

a. Tests of materials from various local sources to determine whether suitable aggregates were present therein and could be obtained and processed economically. The test results showed that there could be made available such material in proportions suitable for an excellent asphaltic concrete mix.
b. Tentative design of mix for testing and cost estimating purposes, using such aggregates with 8.5% of asphalt (of steam-refined, paving grade and 50-60 penetration). Later tests were made to establish a like tentative mix but using asphalt of 85-100 penetration.
c. Tests of such properties as ductility, softening point, penetration and flashpoint; thin-film and spot tests of various asphalts; also determinations of values of the asphaltic concrete as concerns density, stability (compressive strength), flow, immersion compression and sustained load or creep tests, on a sufficient number of mixes (with various percentages of asphalt) to permit establishing the optimum mix. The tests and procedures involved were largely those which have been standardized.
d. Extensive tests involving a number of types (including steam or vacuum refined and catalytically air blown) and penetration grades of asphalts, some with additives (including natural and synthetic rubber), all for the purpose of determining the most likely candidates for ultimate use.
e. Slope-flow tests to determine the behavior and characteristics of selected asphaltic concrete mixes under the conditions of various slopes and temperatures. These tests were made in three phases and consumed a great deal of time but were eminently worthwhile.
f. Various special or non-standard tests, to determine other important properties or limitations. The more important of these were tests of ice action; wave action; alternate freezing and thawing, on 4-in. diameter cylindrical specimens; transmission of 'cold', on one of the rectangular slope-flow specimens; contraction due to low temperatures; permeability; flexure, on small rectangular beams at minus 110 deg. F.; and flexibility, on 3-in. thick discs, 18 in. in diameter, at various pressures and temperatures.

The above and other tests led to the firm conclusion that asphaltic concrete could be placed on slopes as steep as 1 on 1.5 and would remain stable thereon even if the temperature of the surface of the material were somewhat in excess of 140 deg. F. However, it was determined that for Montgomery dam the deck slope should be 1 on 1.7. Further it was concluded that, with the materials available at the Montgomery site, best results would be obtained by means of asphaltic concrete containing approximately 8.5% (in terms of weight of dried aggregate) of steam-refined, paving-grade asphalt as binder.

A more detailed description of these tests, together with the test results and interpretations, will be presented in February, 1958, by the authors before the annual meeting at Montreal, Canada, of the Association of Asphalt Paving Technologists, a technical group specializing in asphalt and asphaltic concrete.

The 'job mix' finally selected was essentially the same as that used in the experimental slope-flow tests. The asphaltic concrete aggregates, as processed from a nearby glacial deposit, were limited to a maximum size of 1-1/2 in. The gradation approximated the following:

Size	Per Cent Passing Square-Mesh Screen (Washed Sample)	Size	Per Cent Passing Square-Mesh Screen (Washed Sample)
1-1/2 in.	100	No. 10	47.2
3/4 in.	87.5	No. 40	30.8
1/2 in.	75.9	No. 80	20.6
No. 4	59.2	No. 200	11.8

In order to determine the effect of thermal changes upon a more extensive surface than was available in the laboratory, a 20 x 120 ft. asphaltic concrete test panel, including horizontal as well as vertical joints, was constructed in late October 1955. It was placed upon a well-drained, slightly sloping area not far from the site. This panel consisted of two strips of asphaltic concrete, each of three layers totaling 7 to 8 in. in thickness after compaction. Over a period comprising two winters, careful observations and precise measurements, between pairs of brass plugs set flush with the top surface, disclosed no detrimental surface changes, variations in length or joint movement.

10. Certain Phases of Design

Slopes

In general, dumped rock for a fill assumes naturally a slope varying between 1 on 1.3 and 1 on 1.4, as was later found to be true in the case of the Montgomery fill. It followed that the slopes of both downstream and upstream faces of the dam should not be steeper; otherwise expensive placement of the rock for such slopes by derrick would become necessary, in order that the slopes would be stable.

For the downstream face of the rock fill, an average slope approximately 1 on 1.4, somewhat flatter than the anticipated natural slope, was used for the entire face.

Quite different were the considerations for determination of the slope of the upstream face of the dam, for that slope involves the asphaltic concrete deck. Of course the steeper that slope, the less would be the quantities involved in the construction. For proper placing of the asphaltic concrete, such slope needed to be uniform, i.e., without berms. Moreover, it needed to be such that the asphaltic concrete thereon would remain stable and not flow downslope even under the highest ambient temperatures. Also the slope needed to be sufficiently flat to permit working on the partially finished asphaltic concrete deck without requiring such facilities as walkways, ladders, etc. The tests referred to at item 9 above had shown that asphaltic concrete would remain stable on a slope as steep as 1 on 1.5. Safety of movement of persons on the slope and facility of machine placement of the asphaltic concrete were believed to be attainable with a slope of 1 on 1.7.

After taking all of the foregoing into account, the latter slope was selected for the upstream face of the dam. As to the safety aspect, construction at that slope proved to be entirely feasible. However, ropes anchored at the crest of the dam were used by contractor's employees to permit greater speed and less effort in moving about on that slope.

Camber

All rock-fill dams settle during and for a considerable time subsequent to construction. Usually, to provide enough height of fill to offset future settlement, an over-fill is prescribed. Without this, settlement might result, not merely in reduction of freeboard but also, in an unsightly sagging appearance, especially at or near the section of maximum height. For Montgomery dam, such over-fill was 3 ft. at the maximum section of the dam, reducing uniformly to zero at each abutment. (See item 12 hereof.)

As a result of this camber, the upstream slope of the dam, as initially constructed, became slightly steeper than 1 on 1.7 at the section of maximum height, viz., 1 on 1.66, flattening gradually to 1 on 1.7 at the abutments. The downstream average slope, intended ultimate to be 1 on 1.4, was thus actually constructed at 1 on 1.36 at the maximum section of the dam.

Cut-off Wall and Its Connection to Asphaltic Concrete Deck

The cut-off wall extending into the foundation rock was located, not at the intersection of the upstream slope of the dam with the foundation rock but instead, a short distance upstream from that intersection, so that the several parts of that wall would be straight in plan. Thus, the horizontal projection of the downstream face of the cut-off wall is a polygonal envelope of the highly irregular line of intersection of the upstream slope of the rock fill with the surface of the foundation rock.

The wall itself is a simple Portland cement concrete structure, reinforced, rectangular in cross-section, 3 ft. wide, extending 3 to 8 ft. into rock, depending on conditions after blasting, and projecting a few feet above the foundation rock, the precise amount depending on the irregularities of the surface of that rock. Although the slope of the top of this cut-off changes from place to place, no steps occur from one part to the other, i.e., there is a continuous surface from abutment to abutment.

For connection of the asphaltic concrete deck to the top of the cut-off, the design was such that the top of the fill, i.e., the base for the asphaltic concrete, would have a transition from its prescribed slope (of 1 on 1.7) to horizontality, flush with the top of the cut-off wall. (Fig. 4)

The asphaltic concrete, curving with the surface of its rock-fill base, was placed upon and over the top of the concrete cut-off wall and extended horizontally a few feet upstream therefrom. The theory of this method of construction is that, by reason of the open nature of that rock fill which is just downstream from the cut-off wall and under the asphaltic concrete, the water

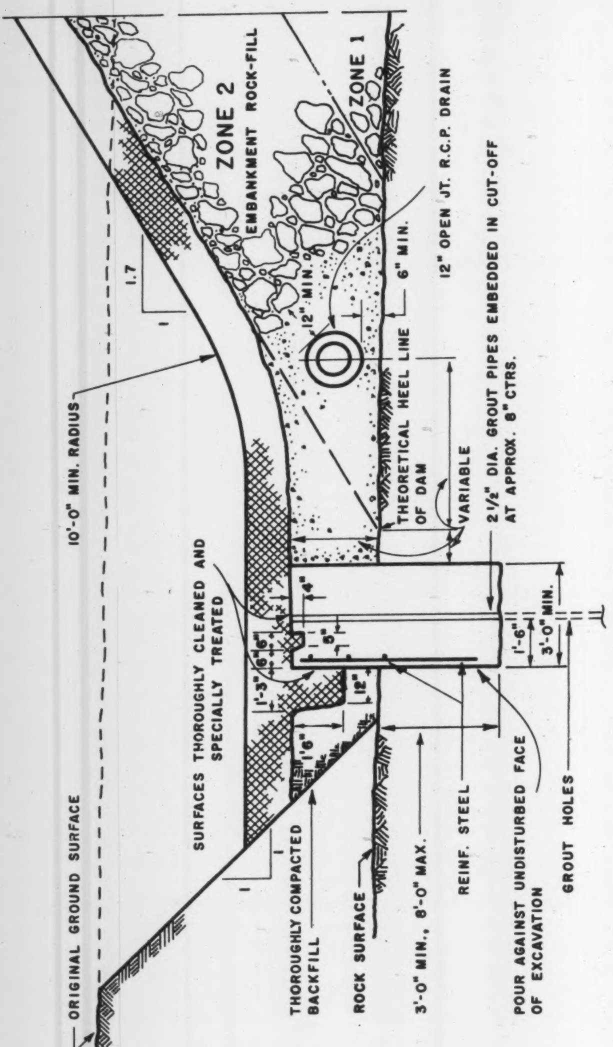
ORIGINAL GROUND SURFACE
10'-0" MIN. RADIUS
SURFACES THOROUGHLY CLEANED AND SPECIALLY TREATED
THOROUGHLY COMPACTED BACKFILL
ROCK SURFACE
3'-0" MIN., 8'-0" MAX.
REINF. STEEL
POUR AGAINST UNDISTURBED FACE OF EXCAVATION
GROUT HOLES
ZONE 2
EMBANKMENT ROCK-FILL
ZONE 1
12" OPEN JT. R.C.P. DRAIN
12" MIN.
6" MIN.
THEORETICAL HEEL LINE OF DAM
VARIABLE
2 1/2" DIA. GROUT PIPES EMBEDDED IN CUT-OFF AT APPROX. 8" CTRS.
1.7
1
1
1
1'-3"
6"
6"
4"
5"
12"
1'-6"
1'-6"
3'-0" MIN.

FIG. 4.—DETAIL AT CUT-OFF

pressure on, together with the dead weight of, the asphaltic concrete and proper bonding will result in a watertight connection between cut-off wall and asphaltic concrete deck.

As an additional precaution, an asphaltic concrete "hook", approximately 12 x 18 in. in cross section, was constructed just upstream from the cut-off wall, so as to extend from the deck asphaltic concrete down into compacted backfill. This increases the area of asphaltic concrete bonded to the Portland cement concrete cut-off wall and thereby further diminishes the chances of leakage between deck and cut-off wall.

Drainage

Although the dumping of loose rock, the usual construction method for such rock fill, is ordinarily assumed to result in free drainage, there are possibilities that such may not everywhere be the case and that there may exist circumstances under which ensurance as to free drainage is of primary importance.

The sluicing water, used in placement of the rock fill, drains down to the foundation of the dam and thence flows off; thus it is possible that the finer material moved by the sluicing water may fill the voids of some lower portions of the rock fill. Although such filling of the lower voids can do no harm as to stability or settlement of the fill, it tends to create within the rock fill an interior barrier which extends to uncertain height and could become impervious.

Consequently, if the reservoir formed by the rock-fill dam is to be emptied from time to time, there exists the possibility that water from leakage or seepage, whether through the deck or in the form of groundwater from the foundation, will pond within the rock-fill voids between the deck and such interior barrier and thus, when the reservoir is drawn below the level of such pond, will exert pressure against the lower side of the deck.

The Montgomery reservoir is intended to be drawn down once a year. Therefore, to eliminate the possibility of such back pressure against the asphaltic concrete deck. there has been provided, below this deck, drainage which makes reasonably certain that such back pressure can not occur. The provision in point consists partly of prescriptions as to the rock to be placed immediately under the deck and partly of a positive drainage system.

The rock-fill volume was divided into two zones. For Zone 1, by far the larger part of the fill, the material was permitted to contain up to 10% of rock fines (4 in. and smaller). For Zone 2, lying between Zone 1 and the asphaltic concrete deck and having a horizontal width of not less than 10 ft., the material was prescribed to be completely free of any and all rocks of 3-in. and smaller size.

The positive drainage consists of an open joint, concrete, drain pipe of 18 in. diameter, installed at the bottom of Zone 2 along the entire cut-off. Near the lowest point of that longitudinal drain, a like connecting, transverse pipe, of 24-in. diameter, provides a drainage outlet into the river bed downstream.

Spillway

For so small a drainage area (8.5 sq. mi.), at so great an altitude (10,861 ft. above M.S.L.), determination of spillway provision is not subject to the normally controlling considerations.

Although rains of day-long duration are known to have occurred, day-long hard rains in that region are unknown. The gated spillway of the nearby Green Mountain dam of the Bureau of Reclamation, located on the Blue River on the westerly side of the Continental Divide, is understood to provide for a peak discharge of 25,000 cfs. from a drainage area of 583 sq. mi. On the assumption that flood peak discharge varies with the square root of the drainage area, the comparable flood peak for the drainage area at Montgomery dam would be approximately 3,000 cfs.

Usually, for ungated spillways, the rise in reservoir level accompanying flood discharges reduces substantially the peak discharge over the spillway as compared with the peak flow into the reservoir, but no substantial reduction in this respect will occur at the Montgomery reservoir, for the reservoir area at spillway crest level is less than 100 acres. A discharge of 3,000 cfs. passing over the spillway would cause the reservoir level to rise 4 ft. to elev. 10,865, or 3 ft. below the crest of the rock fill (exclusive of residual camber). However, there is a parapet, 2 ft. 2 in. high along the upstream edge of the crest, constructed of welded, corrugated steel plates and held vertical by means of steel posts anchored firmly in precast Portland cement concrete blocks which in turn are embedded in the rock fill. The parapet has its bottom edge embedded in the top of the asphaltic concrete deck of the dam. Thus, with such discharge, freeboard and hence margin of at least 5 ft. are provided above that flood level.

11. Construction (Except Deck)

Construction of Montgomery Dam was begun in August, 1854. In its entirety it required a little over three years, not due to requirements and difficulties of the job but, because the annual construction season is so very short.

The rock fill proper was begun at the start of the 1955 construction season and completed at the end of the 1956 season. The asphaltic concrete deck, together with minor other work, was constructed during 1957. Of the total calendar period, hardly one-half involved actual construction.

Quarrying and Processing of Rock for Fill

The contract specifications permitted use in the fill of rock up to 5 tons (Fig. 5). On the other hand, they limited the amount of rock fines (4 in. and smaller) to be used in the fill (see under item 10, Drainage). It was found that upon blasting the granite there resulted fines exceeding the 10% limitation. Although selection of material in loading of trucks by the quarry shoven made it possible to bring the loaded rock within the specification, the residual material was generally useless without some form of screening.

During the early stage of construction, the contractor had installed a trommel or rotating screen, largely because of needs for rock of the smaller sizes, notably to embed open-joint drain pipes (see item 10 hereof) and for preparation of the upstream face of the rock fill as a base for the asphaltic concrete deck (see item 13), but also because the material for Zone 2 was required to be completely without fines. However, the use of the trommel to provide, from the residual quarry material, usable rock for Zone 1 of the fill proved entirely too slow and too expensive.

Next a 'grizzly' or bar screen, of steel rails, was tried and found wanting.

It was not until, in 1956, a so-called wobbler screen had been installed that the job could proceed at the required rate and with assurance that the fill material would meet specifications, even if this involved some double handling of rock. For description of the wobbler screen and its operation, see Engineering News-Record, November 15, 1956, p. 44, and Western Construction, December, 1956, p. 48.

Rock Fill

The rock fill was placed in lifts which were prescribed not to exceed 40 ft. in height. Actually the maximum approximated 30 ft. The contractor was left practically free to work out his own plans in this respect.

Essentially all of the rock was placed by dumping. During and immediately after the dumping of each truck load, the rock was liberally sluiced with water. Each nozzle stream was prescribed to be capable of delivering 500 gpm. at a pressure (measured at nozzle base) of at least 60 pounds per sq. in. (psi.). The sluicing quantity was prescribed to be at least twice that of the gross volume of the rock placed and so sluiced.

Largely the contractor was left free in the choice of method to construct the upstream slope of the dam (1 on 1.7), being flatter than the natural slope of dumped rock. Generally this was accomplished by a temporary overfill at the natural slope, whereupon such overfill at the top of the lift was moved down slope to make up for underfilling at the bottom of the lift.

Grouting

The grouting through and along the cut-off wall at Montgomery dam, although a part of the general contract, was sublet to a company specializing in that type of work.

The grout curtain extends vertically 150 ft., below the base of the cut-off wall, across the deeper part of the valley, decreasing to 100 ft. at the abutments. Grouting was prescribed to take place at the maximum pressure attainable, not in excess of 200 psi. Whereas this prescription was somewhat severe for a dam not over 113 ft. high, it took into account the fact that the grouting would have to serve as well in case of future raising of the dam by some 50 ft.

The common procedure was followed in that, first, the upper 50 ft. of the curtain were grouted by means of primary holes plus intermediate or secondary holes where necessary. In general, in the upper 10 ft. of rock grouting pressures of 75 psi. were attained without uplift of cut-off wall. Some leakage of grout to the surface occurred, but, if any grout entered nearby rock fill, it was not observed.

Thereupon grouting at lower stages of the grout curtain took place. This was discontinued near the end of the 1956 construction season, although in places there was indicated additional work which would have improved the curtain. Termination was deemed by the City to be appropriate in view of the limited purpose of grouting at a deck-type, rock-fill dam (see item 6).

12. Settlement of Fill

All rock-fill dams will settle during construction and further still upon completion. Presumably the reason is that, due to increasing load, the individual pieces of rock bearing upon each other will become blunted at their contact points. Measurements made at rock-fill dams show that this settlement process continues at a diminishing rate over several years.

Settlement is affected by various aspects of construction but, assuming reasonable uniformity of construction, settlement may be expected to vary about directly with height of fill.

Besides the essentially vertical settlement due to dead weight, there may be expected additional settlement, vertically and as well horizontally in a downstream direction, upon filling of the reservoir and the loading of the deck and hence the dam as a whole. As is customary, the design of Montgomery dam provided a camber, in this case 3 ft. maximum (see item 10).

The Montgomery reservoir has not yet been filled; hence settlement data to date are no more than preliminary, being due only to the gravity effect on the fill. At the section of maximum height of completed fill, settlement measured during June 1957 (i.e., at the beginning of the year's construction season), as compared with the elevation of the same point in October 1956, was only 0.045 ft. or less than 1 in. At no location along the entire dam did the settlement over that period exceed 0.055 ft.

13. Preparation of Base for Asphaltic Concrete Deck

It was recognized that the surface of the sloping upstream face of the rock fill, upon which the asphaltic concrete deck was to be placed, must among other things be sufficiently stable to support the paving equipment during the placing of the first layer of asphaltic concrete.

The asphaltic concrete deck was prescribed to rest essentially on Zone 2 material (minimum horizontal thickness, 10 ft.) but it was specified that over that material there should be spread 3/4 - 3 in. rock, placed in layers and compacted in such a manner that the larger interstices at the surface of the Zone 2 material would be filled and the surface consolidated (Fig. 6). The objective was to avoid local settlement during subsequent operations of placing and rolling the asphaltic concrete. A further objective was a fairly smooth surface, free from any substantial amount of undulation or unevenness.

Near the end of the 1955 construction season, during which a portion of the top surfacing material was placed, serious doubt arose as to whether subsequent compaction alone would result in a stable surface. It was concluded that a 'penetration' coat of an appropriate asphalt binder might be required to be applied to the so-consolidated base just prior to the placing of the asphaltic concrete deck.

No surface consolidation of the base was attempted until near the end of the 1956 working season, when the rock fill was nearing completion and the major portion of the 3/4 - 3 in. surfacing material had been placed. After some experimenting, it was thought that consolidation by several upslope passes of a vibratory roller probably would afford adequate compaction (Fig. 7). However, due to the fact that winter closed in during September of that year, some of the intended work was deferred, to be done during the next construction season. It was realized that in all probability the effects of winter would require some refinishing, at least of parts of the surface, prior to placing the asphaltic concrete deck during the summer of 1957.

Accordingly, it was arranged that the contractor for the asphaltic concrete deck should do any repair and supplementary work upon the base surface, accomplish further compacting of that surface, apply a penetration coat of

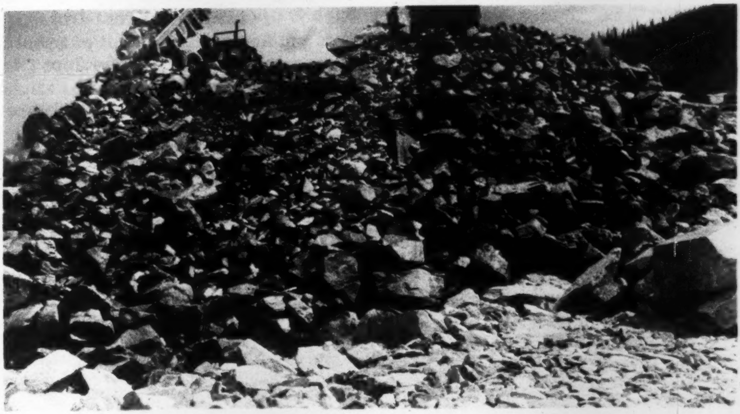

FIG. 5.—TYPICAL ROCK FILL

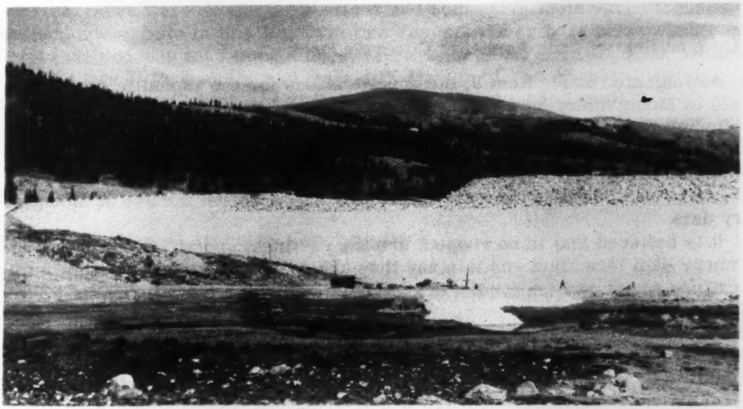

FIG. 6.—PARTIALLY COMPLETED BASE FOR ASPHALTIC CONCRETE DECK, SLUICING MONITORS BARELY VISIBLE AT FOOT OF ADVANCING FACE OF ROCK FILL

asphalt and do whatever else might be necessary to complete the base suitably to receive the asphaltic concrete deck. Such further work is described at item 15 hereof.

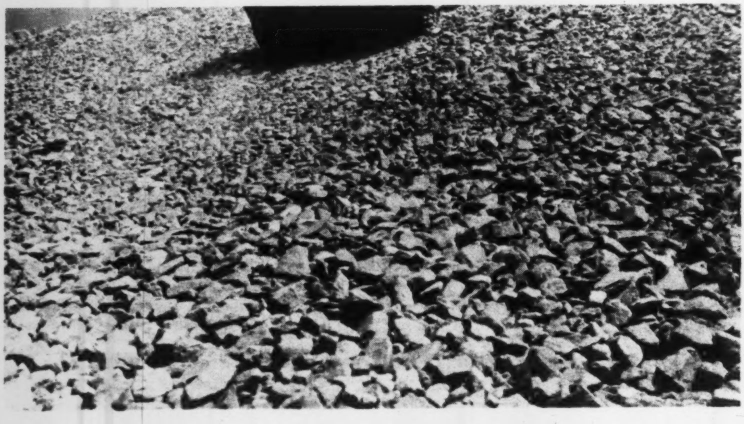

FIG. 7.—VIBRATORY ROLLER COMPACTING SURFACING MATERIAL

14. Specifications for Asphaltic Concrete Deck

As indicated under item 9, the specifications for the asphaltic concrete deck of Montgomery dam reflected the results of experimentation. Also they reflected the influence of pertinent parts of what were considered the best of the latest specifications for asphaltic concrete pavements of turnpikes and other important highways. However, in some important respects, even the requirements of the latter were materially strengthened for use at Montgomery dam.

It is believed that in no respect are the pertinent specifications for Montgomery dam less rigid and in many they are more rigid than any other specifications so far written for the design and placing of a hot bituminous mixture. For instance, asphalt technologists have been somewhat concerned because of changes known to take place in certain of the physical properties of asphalt when thin films thereof are subjected to relatively high temperatures. This condition is approached when the asphalt is introduced (in some instances, by spraying under pressure) into and mixed with a mass of hot aggregate. The result is an ageing action, normally evidenced by loss of weight, increase in softening point and decreases in ductility and penetration. Most specifications for asphalt itself had not embodied (for acceptability) any limitations upon such changes.

However, the Montgomery specifications, by prescribing limits upon the permissible changes in properties of the residues as the result of thin-film tests (oven heating at 325 deg. F. for 5 hours), took cognizance of the foregoing facts. The specifications were intended to confine the selection of asphalts to those exhibiting small, yet reasonable, changes in those critical properties.

Despite the disadvantage of appearing to be unduly complicated, there were included in the specifications for asphaltic concrete a number of other largely new requirements aimed to secure, under the particular nature of the work, results of the highest quality consistent with reasonable cost. Such included the prescribing of -

> Automatic recording, at a single central location, of all important temperatures involved in the operation of the mixing plant.
>
> Unusually close control of conditions of placing, including temperatures and moisture—this because, even in summer, the temperatures are in a relatively low range.
>
> A high degree of density of the asphaltic concrete in place—despite such difficulties of placement as result from the relatively steep slope.

Pertinent in this connection, although not required by the specifications, is the fact that, in order to limit the extent of fractured faces, all boulders larger than 6 in., as found in the glacial deposit, were wasted. The processed material was supplied to the cold-feed bins in 3 sizes, to wit, 1-1/2 to 5/8 in., 5/8 in. to No. 4 and minus No. 4.

The specifications provided that, where needed, there be applied between asphaltic concrete surfaces a 'tack' coat of paving-grade asphalt or rapid-curing, liquid asphalt, in either case to be of standard grade as established by The Asphalt Institute. The rate of application was prescribed not to exceed 0.1 gal. per sq. yd.

For the drafting of the specifications and in order to obtain the benefit of the most advanced thinking in the design and construction of such asphaltic concrete, there were sought suggestions and comments from various qualified persons, specialists in this field. In this respect the authors acknowledge especial indebtedness to, among others, Messrs. John M. Griffith, A. S. Wellborn, the late J. R. Banning and Donald D. Dagler of The Asphalt Institute, the headquarters of which are at College Park, Maryland; Jewell R. Benson, Consulting Engineer, Denver, Colo.; L. M. Ellsperman of the Bituminous Laboratory, Bureau of Reclamation, Denver, Colo.; J. O. Izatt, of the Shell Oil Company, New York; and R. A. Andreasen and R. F. Jouannet, of the Barber-Green Co., Aurora, Illinois.

15. Construction of Asphaltic Concrete Deck

Bids on the construction of the asphaltic concrete deck having been received in November 1956, the contract was awarded before the end of that year. Thus during the winter the successful bidder had opportunity to study the requirements of the job and to plan and provide appropriate equipment.

Except for modifications to paving equipment and trucks and in the methods of handling the same, in order to secure efficient operation on the sloping upstream face, paving methods and techniques were those normally required for high-grade, hot-mix, asphaltic paving of highways and aviation runways. The contractor already owned an appropriate mixing plant and appurtenant facilities, so that it was necessary only to supplement the same in certain respects, including provision for thermal control and other special equipment required under the specifications.

However, the need for modifying existing highway paving equipment and providing some entirely new forms of equipment was more acute as concerns provision for operating heavy equipment on the upstream slope (1 on 1.7) of the dam, i.e., in the placing and compacting of the asphaltic concrete deck. In this connection the contractor designed and constructed a novel turntable, to be located upon and moved, by caterpillar tractors, along the 20-ft. wide roadway on the crest of the rock fill. This was designed to transfer the paving machine from one lane (up and down slope) to another and, by permitting the requisite change in direction, to transfer trucks (carrying asphaltic concrete) from the roadway on the crest to and from the sloping deck. The turntable was supplied with electric powered, variable speed drums and winches so that, by movement of the loaded and empty trucks down and up slope independently of the movement of the paving machine, there was permitted largely uninterrupted paving from bottom to top of the slope (Fig. 8).

The specifications permitted much latitude as to type of roller to be used in order to attain the required density of the asphaltic concrete. After some experimentation, the contractor was successful in adapting a standard type of vibratory roller to the needs of the case. The movement of the roller up and down slope was powered and controlled by a caterpillar tractor operating on the crest roadway and equipped with a boom and cables, leading from its power take-off to the roller.

As mentioned at item 13, it was arranged that the contractor for the asphaltic concrete deck should do certain work which had been deferred from the close of the previous (1956) construction season. The most important of the items involved in this respect was that of completing the attainment of a surface or base capable of supporting and remaining stable under the weight and operating movement of the paving equipment. The effect of movement of the 11-ton paving machine was of course greatly aggravated by reason of the slope.

The situation as regards completion of such preparation of the base became the more acute because of the unexpected delay in the melting of the winter's heavy accumulation of snow and in the arrival of temperatures permitting the work to proceed.

It had previously been concluded that the surface of the rock base would need to be treated with an appropriate penetration type of asphalt. After an additional pass of the vibratory roller over the entire surface, such a penetration coat was applied.

Even after such application, however, the surface still showed insufficient stability. After consideration of several expedients which seemed feasible, it was concluded to place upon the base what was referred to as a 'leveling course' of asphaltic concrete, of the same mix as specified for the deck proper. This was intended to be relatively thin. Because the sloping base would not yet tolerate the movement of the paving machine without disturbance of the surface, resort was had to placing of the leveling course by means of trucks.

For this operation, trucks loaded with the hot asphaltic concrete were backed down the slope, by means of the cable and winch controls located at the turntable. During such lowering along the slope, the partly open tailgate of the trucks permitted spreading of the material ahead of the backing truck. This spreading had to be supplemented by hand work. Subsequently the so-spread material was subjected to a single pass of the vibratory roller.

The leveling course varied in thickness from less than 1 in. to about 3 in. It resulted in a base upon which the paving machine and supply trucks could

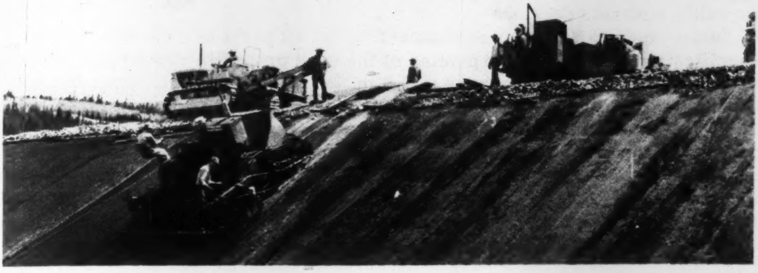

FIG. 8.—SOME OF THE EQUIPMENT INVOLVED IN THE CONSTRUCTION OF ASPHALTIC CONCRETE DECK

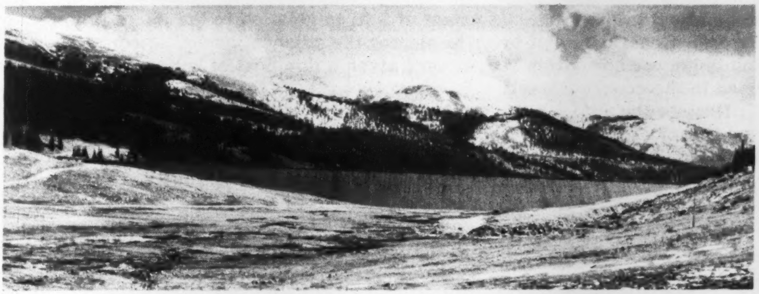

FIG. 9.—UPSTREAM VIEW OF DAM

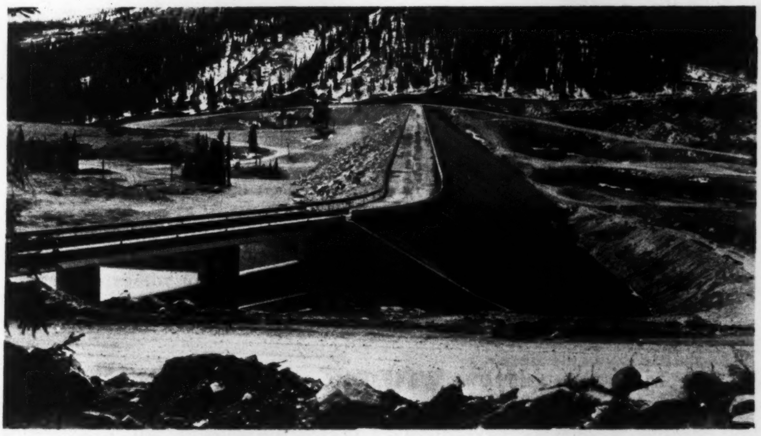

FIG. 10.—VIEW ALONG DAM

and did operate to place the first regular and of course subsequent layers of asphaltic concrete deck.

Subject only to adequacy of compaction "of all parts of each layer", the specifications permitted the placing of the deck proper in two layers. The minimum permissible thickness was 3 in. (after compaction). However, the contractor, wisely it is thought, chose to place the deck proper in three layers.

In this connection, there arose the question as to what value should be attributed to the leveling course in terms of an equivalent portion of thickness of regularly placed and compacted deck. Except for special conditions obtaining near the north end of the deck, it was concluded in behalf of the City in effect that it would be appropriate to allow 1-1/2 in. as much equivalence of the leveling course. The result was a total machine-placed, asphaltic concrete thickness approximating 10-1/2 in. This was placed in layers having thicknesses approximating successively 4 in., 3-1/2 in. and 3 in., all measured after compaction.

The asphaltic concrete was placed on the upslope pass of the paving machine in 12-foot widths or strips. All joints within the layers were required to be offset to the minimum extent of 3 ft. in relation to the joints in lower layers. Generally, just prior to placing the adjacent asphaltic concrete, cold joints between or within strips were given a thin coat of hot asphalt of the type used in the concrete itself.

Despite the unfortunate lateness in opening of the 1957 construction season and therefore its brevity and despite the complication in attainment of a stable base for the regularly placed asphaltic concrete of the deck, the completion of the deck was accomplished by the time weather conditions were becoming intolerable.

For over-all views of the completed structure, see Figs. 9 and 10.

16. Cost Data

The more important bid comparison figures of the successful bidders are set forth below:

Rock fill
  Zone 1 material, 627,000 cu. yds. @ $1.20
  Zone 2 material, 78,000 cu. yds. @ $3.00
  (Average for entire rock fill — $1.40 per cu. yd.)

Grouting
  Drilling with AX bit, 3,000 ft. @ $5.85
  Drilling with BX bit, 26,000 ft. @ $7.15
  Grouting, 7,500 sacks of cement @ $6.00

Preparation of rock base for asphaltic concrete deck
  By dam contractor, 27,000 sq. yds. @ $1.60
  By deck contractor, supplemental, 2,000 tons of rock of prescribed sizes $10.00
  Asphalt penetration coat, 20,000 gals. @ $0.35
  (Average, for entire deck area - $2.60 per sq. yd.)

Asphaltic concrete deck
  Asphalt, alone, 1,400 tons @ $43.00
  Asphaltic concrete, in place, except asphalt, 19,000 tons @ $11.00
  Vibratory rolling, 60,000 sq. yds. @ $0.25
  (Average, for entire deck - $10.50 per sq. yd.)

The total construction cost of the dam, including exploration, was $2,663,910.

17. Personnel Involved

Throughout the development of the project of which Montgomery dam is a part, the city's pertinent activities have been receiving the general direction of J. B. Biery, City Manager. During the entire period, the Superintendent of the Water Division has been J. S. Nichols. The Water Division operates as part of the Department of Public Utilities, of which initially T. M. Hohl was Manager and more recently, during the major part of the construction, R. D. Nixon was the latter's successor.

The engineering aspects of the development of the project of additional water supply and the construction of Montgomery dam have throughout been in charge of Black & Veatch, Consulting Engineers of Kansas City, Missouri, with N. T. Veatch, M. ASCE, in general supervision and G. R. Scott, M. ASCE, in charge of the project. Throughout the performance of the contract for the construction of the rock fill, 1954-56, Martin Hemker served as Resident Engineer for Black & Veatch; for the construction of the asphaltic concrete deck in 1957, that position was occupied by William Gus Koulas, A.M. ASCE.

On exploration for dam and reservoir site, design and preparation of specificiations for rock fill, grouting and asphaltic concrete deck and construction of those elements of the work, the office of F. W. Scheidenhelm, M. ASCE, including the co-authors of this paper, served as consultant to Black & Veatch.

During 1957, in connection with the construction of the asphaltic concrete deck, L. M. Ellsperman contributed information acquired during the Glen Anne dam construction.

The Bituminous Laboratory of the Bureau of Reclamation, located at the Federal Center, Denver, Colorado, and at which there was carried out the extensive pre-construction experimentation, is under the general direction of Walter H. Price, M. ASCE, Chief, Engineering Laboratories of the Bureau, and during that experimentation was under divisional direction by Willis T. Moran. The immediate testing throughout was in charge of L. M. Ellsperman, who since 1954 has been head of the Bituminous Laboratory.

For the State of Colorado, the plans of the rock-fill part of the dam were reviewed and approved by M. C. Hinderlider, State Engineer. Review and approval in the case of the asphaltic concrete deck were by his successor, J. E. Whitten.

In connection with the actual construction of the deck, the laboratory of the Colorado Department of Highways, of which E. G. Swanson is Staff Materials Engineer, did the major part of the pertinent testing.

The contractor for the rock-fill dam was Fisher Contracting Company, Phoenix, Arizona, with Ray Spangler in 1955 and Ken Willis in 1956 as job superintendents. Grouting was sub-contracted to Continental Drilling Co., of Los Angeles, California, with Bond H. Jobe as field superintendent.

The contractor for the construction of the asphaltic concrete deck was San Ore Construction Company, of McPherson, Kansas, with Dean Dillon as job superintendent.

18. Significance of Montgomery Type of Dam

In general, where rock suitable for such fill is readily available, as at Montgomery, the combination of rock fill and asphaltic concrete deck not only makes maximum use of locally available material but does so at a substantial saving in cost as compared with other types of dam offering equal advantages. Also it permits economical dam construction in areas where there is not available material either for a dam or core of the rolled-earth-fill type or for a dam involving Portland cement concrete but there is available rock good enough for a rock-fill dam.

In general, as compared with decking of steel or Portland cement concrete for a rock-fill dam, the combination in point results in substantial economy. (This was true at Montgomery despite the fact that all materials required for Portland cement concrete excepting the cement itself were available locally.) In addition, there obtain the other advantages pointed out under items 6 and 7 hereinabove.

Although apparently without precedent, at least in the United States, an asphaltic concrete deck of the Montgomery type is believed to have economic possibilities also for situations where material suitable for rock fill is not available but there is available permeable material, such as gravel, disintegrated rock or earth, not adaptable to a conventional rolled fill. Such construction would obviate the need for watertightness of the fill. There would be required compaction only for the purpose of reducing, if not minimizing, settlement. However, such a form of dam would involve important design and construction considerations not involved in a rock-fill dam. As stated in item 5, a modified earth-fill type of construction was considered for the Montgomery site but, without study for detailed design, was found not to indicate sufficiently greater economy as compared with the constructed type of rock fill with asphaltic concrete deck.

The authors have found that, in the case of a hydroelectric development scheduled for construction beginning in 1958 and involving a dam twice as high as that at Montgomery, the project, when appraised on the basis of sound economic criteria, has become feasible by reason of the use of a dam of the rock-fill type (with asphaltic concrete deck), whereas with any other type of dam it would not be feasible.

DISCUSSION

G. R. SCOTT,[4] F. ASCE.—The authors' comprehensive description of the development of this project leaves nothing of importance to add relating to the design of the structure. This discussion is submitted primarily to record the behavior of the dam with regards to settlement and horizontal movements after having been filled two times and to describe briefly, the effect on the asphaltic concrete membrane of one complete cycle of filling and emptying of the reservoir.

The rock fill was completed in October 1956, the asphaltic concrete membrane was placed during the summer of 1957 and the reservoir was filled in 1958. It was emptied during the winter and was filled again by early July in 1959.

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[4] Black & Veatch Cons. Engrs., Kansas City, Mo.

TABLE 1.—SETTLEMENT DATA FOR MONTGOMERY DAM

Station	From Oct. 1, 1956 to June 10, 1957	From Sept. 24, 1957 to Aug. 20, 1958	From Aug. 20, 1958 to July 1, 1959
7+95	0.03 ft	0.06	0.06
9+45	.04	0.12	0.04
10+95	.05	0.15	0.05
12+45	.05	0.16	0.07
13+95	.05	0.18	0.03
15+45	.05	0.16	0.04
16+95	.04	0.10	0.04
18+45	.05	0.03	0.01
19+95	.04	0.00	-0.01[a]
21+45	.04	0.01	0.00

[a] Represents a raising above the preceding year.

TABLE 2.—HORIZONTAL MOVEMENT OF CREST OF MONTGOMERY DAM

Station	From Sept. 24, 1957 to Aug. 20, 1958		From Sept. 24, 1957 to July 1, 1959	
7+95	0.10 ft	U	0.08	U
9+45	0.10	U	0.08	U
10+95	0.02	U	0.16	U
12+45	0.02	D	0.06	U
13+95	0.00	-	0.07	U
15+45	0.00	-	0.06	U
16+95	0.06	U	0.05	U
18+45	0.00	-	0.08	U
19+95	0.05	D	0.11	U
21+45	0.00	-	0.05	U

U = Upstream movement
D = Downstream movement

Bench marks were established on the crest of the dam after the completion of the rock fill in October, 1956 and levels were run again in June of the following year to measure the amount of settlement during the winter. These bench marks were destroyed during the installation of the asphaltic concrete membrane in 1957, but others were established at the conclusion of the work from which subsequent measurements have been made. These final bench marks were not at the same locations as the earlier ones and the settlement values of June 10, 1957 shown in the tabulation have been adjusted from the original measurements to conform with the stations indicated.

The horizontal movements have been measured from a series of reference points on the tops of the steel posts supporting the upstream parapet. Obviously such points are not ideal for the purpose of measuring the true horizontal movement of the dam as they will reflect as well the amount of any tilting experienced by the post. However, the fact that the measured displacements have been small and that the upstream wall is not distorted is evidence of very little horizontal movement of the dam itself.

The bench marks are designated by station numbers along the axis of the dam. Station 5+00 is at the extreme south end of the dam. The bench mark at 7+95 is at a point where the fill is 62 feet deep; the point of maximum fill (113 feet) is about station 13+00 and the depth of fill at station 21+45 is 35 feet. The depths of fill at the other bench marks may be approximated by assuming uniform slopes between the end bench marks and station 13+00.

The results of the measurements are given in the Tables 1 and 2. The measurements made in 1958 and 1959 were made at practically full reservoir.

A careful examination was made of the surface of the asphaltic concrete in 1959 before filling for the purpose of estimating the extent (if any) of damage due to ice and to locate and repair any cracks which might have opened due to settlement or contraction.

Apparently, the effect of ice on the surface of the pavement was negligible because the surface sheen caused by the compacting roller appeared fresh and at no place was it noticed that any of the surface had been pulled loose by adherence to the ice as it must have broken away and dropped with the water level during the winter.

Several short horizontal and vertical cracks were observed in 1959, before the reservoir was refilled. All were above spillway level (El. 10,861) and varied from 1/4 to 1/2 in. wide and were about 4 in. deep, or through the top layer. The horizontal cracks ranged from 2 to 6 ft long and the vertical cracks were about 10 ft long, following construction joints.

The cracks were cleaned out and coated with hot asphalt, then filled with sand and the whole covered with hot asphalt. The asphalt had a penetration at 77 F° of 120-300, fluxed or blended with a kerosene type solvent.

J. L. SHERARD,[5] M. ASCE.—The authors have performed a valuable service in summarizing for the profession the problems which they encountered in the design and construction of Montgomery Dam. The use of a 12 inch thick asphaltic concrete upstream slab as a water barrier on a rockfill dam of this size is a considerable break with past practice.

The writer's first reaction when hearing several years ago that this dam was to be constructed to an eventual height of over 150 feet with an asphalt upstream face in the high Rocky Mountains was incredulity. This has changed gradually to respect for the foresight and the courage demonstrated by the engineers who have successfully completed this very interesting structure.

As a result of their investigation, the authors concluded "From the evidence available, there is good reason to believe that asphaltic concrete, when properly designed, placed and compacted, will be at least as durable as competitive materials for such deck construction." While many engineers will be skeptical of this statement, their skepticism would be lessened appreciably if they had the opportunity to examine personally a sample of the compacted asphaltic concrete used at Montgomery Dam. The writer had an opportunity to examine a 4 inch diameter core which had been cut from the compacted asphaltic concrete probably from one of the test sections. This suffered little by comparison with ordinary Portland cement concrete in density, hardness and apparent durability.

In their comparison between decks of asphaltic and Portland cement concrete, the authors discuss the fact that a upstream zone of hand-placed or crane-placed rock would probably have been used under a concrete slab. The writer can see no necessity for preparing a better base for a slab of reinforced concrete than would be required under the asphaltic concrete slab used.

One of the features of the project which probably had an influence on the decision to use asphaltic concrete, in spite of the fact that there is no direct precedent for it, was the fact that the dam retained a relatively small reservoir which would be emptied each year during normal operation. Also with a 6 foot diameter outlet conduit, it is possible to lower the reservoir level fairly rapidly in the event that an unexpectedly large leak developed. From this standpoint, it seems that the situation was favorable for the trial of the asphaltic concrete.

The authors point out that the 1.7:1 upstream slope used was primarily governed by the properties and construction problems of the asphaltic concrete deck. Laboratory "slope-flow" tests led to the conclusion that the asphaltic concrete could be placed on slopes as steep as 1.5:1 and would remain stable even if the temperature rose to 140 degrees Fahrenheit. It would be very interesting if the authors would describe briefly how these "slope-flow" tests were performed.

It would be interesting if the authors would elaborate in their closing discussion on their thoughts regarding the influence which ice on the surface of the reservoir may have on the performance of the asphaltic concrete deck. How thick an ice sheet is expected? Will this ice sheet bond securely to the upstream deck? If so, what might happen if the reservoir level lowered removing the support while the ice sheet was still thick? The writer has seen relatively large boulders torn out of rock riprap blankets by such action.

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[5] Woodward-Clyde-Sherard and Associates, New York, N. Y.

Even though the laboratory tests indicated the asphaltic concrete would be stable on 1.5:1 slope, the upstream slope was chosen as 1.7:1 to provide a little added factor of safety and to facilitate construction. In the comparison between the costs of upstream decks from Portland cement and asphaltic concrete, computations indicated that the cost of the asphaltic concrete was approximately 50% of that of the Portland cement concrete. If Portland cement concrete had been used, the upstream slope for a dam of this height could have been constructed considerably steeper with a resulting cost advantage in favor of the Portland cement concrete slab.

In order to assure that no hydrostatic pressures would develop behind the asphaltic concrete deck when the reservoir was being lowered, the rock directly beneath the deck (Zone 2) was screened to remove all rocks smaller than 3 inches. As a leveling course and a base for the deck, a layer of finer rock graded between 3/4 inches and 3 inches was used. Fig. 7 shows a vibratory roller compacting this surfacing material. It would be valuable if the authors would describe the average and maximum thickness of this finer material used directly under the asphaltic concrete deck.

As stated in the paper, one of the principal advantages of the asphaltic concrete deck was the fact that it could be constructed much more rapidly than a Portland cement concrete slab. It should be pointed out that an upstream face of steel plate can also be placed very rapidly. During 1958, a rockfill dam, approximately 80 feet high, was constructed by the City of Caracas, Venezuela, for an expansion of their municipal water storage. One of the principal requirements imposed upon the designers of this dam was the fact that it had to be both designed primarily because it could be erected rapidly in wet weather. While its initial cost is higher, the steel plate decking is probably more flexible and better able to follow differential settlements than either asphaltic or Portland cement concrete and also probably has better resistance against the erosive action of waves than asphaltic concrete.

More than a year has elapsed since the authors prepared the paper. It is hoped that in their closing discussion they will be able to give some additional information regarding the performance of the dam. Also in the closing paragraph of the paper, it was stated that a hydro-electric project including a dam similar to Montgomery Dam but twice as high was scheduled for construction in 1958. Further details of this dam would be particularly welcomed by all dam engineers.

F. W. SCHEIDENHELM,[6] JOHN B. SNETHLAGE,[7] F. ASCE, and ARTHUR N. VANDERLIP,[8] F. ASCE.—The authors feel gratified that the various discussions of their paper contained generally favorable comment.

Mr. Scott points out that settlement of the rockfill dam with which he has been concerned, to date, has been gratifyingly low and well within the limits anticipated.

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[6] Cons. Engr., N. Y. (Deceased)
[7] Cons. Engr., East Chatham, N. Y.
[8] Cons. Hdr. Engr., New York, N. Y.

Mr. Scott further reports some cracks and the repair thereof. Just what caused these cracks is not definitely known.

The possibilities suggesting themselves are:

a. Near the crest of the dam, working conditions were not as favorable as along the main portion of the upstream slope, so that slight deviations of the paving machine, caused by inaccurate placing of the hoisting center, may have resulted, here and there, in a weak "vertical" joint between adjacent strips.

b. The paving machine could not lay a strip continuously to the exact top of the slope because, before reaching that top, its position necessarily had to change from a sloping one to a horizontal one. Hence the extreme upper portion of the deck, that is, some 4 ft or 5 ft thereof, had to be paved by hand work, involving cold "horizontal" joints. If such joints were made without the required care, opening at a later date could occur.

c. It is not impossible that the severe cold of the region in which the dam is located could cause weak joints to open due to contraction of the asphaltic concrete. However, for well-made joints the writers consider this latter possibility unlikely, because such behavior was not borne out by the laboratory tests, and the cracks are reported to be not continuous, either in length or in depth.

At any rate, all of the cracks occurred above reservoir full level, that is, within the upper 7 ft of the deck, (measured vertically), penetrated no deeper than the upper layer of the four-layer deck and were easily accessible for repair.

As to leakage, not too much is known with certainty; the location of the dam, practically inaccessible during part of the year, does not permit frequent inspection.

Inquiries directed to the office of J. S. Nichols, Chf. of Operations, Dept. of Pub. Utilities, Colorado Springs, disclosed that, as of June, 1959, "There is no evidence at this time of any appreciable leakage."

Mr. Sherard's discussion touches on the principle of use of asphaltic concrete as a deck material. He states that the rockfill base upon which the asphaltic concrete was constructed could serve equally well for a Portland cement concrete slab. The writers fully agree. However, further on in his discussion, Mr. Sherard, in discussing the economy of these two deck materials, states that the cost of such a Portland cement deck could have been reduced by steepening the upstream face. The writers again agree, but feel compelled to point out that, if the rockfill material itself is to be deposited in the most economical manner, that is, by dumping, the upstream slope should not be steeper than the natural slope of dumped and sluiced rock, approximately 1 on 1.3. Further steepening, to permit economy in the use of a Portland cement concrete, would require the use of an expensive rubble masonry wall to retain the dumped rockfill material to such steeper slopes.

Mr. Sherard inquires about the thickness of the graded $\frac{3}{4}$ in. to 2 in. material immediately underlying the asphaltic concrete deck. The purpose of using this material was to convert the relatively rough face of dumped Zone 2 material into a relatively smooth and even surface upon which, after stabilization, the asphaltic concrete paving machine could be moved to deposit the first layer of asphaltic concrete.

To avoid excessive depth of this finer material, the rockfill specifications suggested (but did not prescribe) that for Zone 2 material the larger rock pieces, permissible in Zone 1 material, be omitted. In a subsequent specification for another dam, the writers prescribed that the rock of the Zone 2 material should not exceed 18 in.; also that the upstream, sloping surface of the Zone 2 material be rolled with a vibratory roller of large diameter.

Thus the depth of this surfacing material varies. The minimum depth can approximate zero, that is, where the rock of Zone 2 is at the prescribed surface. The maximum depth is that of the deepest, irregular surface space between adjacent rock pieces of the Zone 2 material. It is the writers' present offhand impression that such maximum depth should nowhere be in excess of 15 in. and that the average depth, that is, the total volume of surfacing material used, divided by the deck area, would not be greater than 6 in.

The construction item for the surfacing work specified payment per square yard of slope area and hence the actual amount of material required for the surfacing work is not known to the writers.

Mr. Sherard states that a deck of plate steel, as used by him on a rockfill dam in Venezuela, is more flexible than either an asphaltic concrete or Portland cement deck. The writers agree and are of the opinion that flexibility of the deck material is of the greatest importance. However, they wish to point out that an asphaltic concrete deck is much more flexible than a Portland cement deck and that a flexibility greater than that of an asphaltic concrete deck, given reasonably good construction of the underlying rockfill, is probably not required.

Mr. Sherard further inquires about the method of the "slope-flow" tests. The entire testing program, procedure and interpretation of test results was described by the writers in their paper entitled: "Laboratory Investigation of Asphaltic Concrete, Montgomery Dam, Colorado," presented at the annual meeting of The Association of Asphalt Paving Technologists in Montreal, Quebec, Canada, on February 19, 1958. This paper was subsequently published in full in the Proceedings of that Association, Vol. 27, pp. 383-493.

The testing method for the slope-flow tests involved, in essence, the preparation of test specimens, from aggregates and asphalt to be used for actual construction (the latter in varying percentages), prepared in a manner which simulated as much as possible actual construction methods, including a base of compacted crushed stone. These specimens were then subjected, when placed under various slopes, to controlled temperatures from above, as well as from below. Flow measurements were made from markers on the surface of the specimen.

The testing program did include tests pertaining to ice effect, as mentioned by Mr. Sherard. A specimen of asphaltic concrete and a volume of water were frozen solidly together and then tested for breakage. The frozen asphaltic concrete specimen was held rigidly in a hydraulic test machine and breakage effected by application of pressure on the ice. In each case the ice broke along a plane at or practically parallel to the surface of the specimen, without affecting the asphaltic concrete surface in any manner whatsoever.

The dam has, to date, gone through three winters and the owner reports as of May, 1959, "There are no cracks nor checks evident, nor is there any evidence of ice scour or abrasion."

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3079

ROCKFILL DAMS: STEEL-FACED DAM

By James L. Sherard,[1] F. ASCE

SYNOPSIS

Speed of construction and structural integrity were the chief advantages gained by the use of a steel-plate facing on a rockfill dam built in Venezuela in 1958. At present, steel-plate facing is seldom considered for use on earth and rockfill dams, although a number of such dams have been built in the past and have seen many years of satisfactory service. The principal reasons for not considering steel facing the dams have been concern about possible corrosion of the steel and cost. However, the advantages offered by steel-plate impervious membranes for dams are such as to justify the consideration of this type of construction under some site conditions. Experience with structures in the Western United States gives weight to this statement.

RIO LAGARTIJO LOW DAM

In recent years the demand for domestic water in Caracas, Venezuela, has grown considerably faster than anticipated, and a severe shortage developed in 1957. During 1957 and 1958, water had to be severely rationed and a supplemental supply had to be brought into the city in tank trucks from sources outside the system. The cost of the supplemental water and the inconveniences

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Note.—Published, essentially as published here, in October, 1959, in Civil Engineering, Vol. 29, No. 10, pp. 698-701. Positions and titles given are those in effect when the paper was approved for publication in Transactions.

[1] Partner, Woodward, Clyde, Sherard & Associates, New York, N. Y.

caused by the shortage led the Instituto Nacional de Obras Sanitarias (INOS) to accelerate the rate of construction of new storage facilities.

Previously planned for construction in the early 1960's was a large reservoir to be formed by an earthfill dam 164 ft high on the Rio Lagartijo, about 40 miles southeast of Caracas. The size of this dam and the incomplete state of the design studies made it impracticable to attempt its completion in time to alleviate the water shortage anticipated during the 1959 dry season. Consequently it was decided to proceed immediately with the construction of a low dam on the Rio Largartijo some 3,300 ft upstream from the site chosen for the main dam. The low dam was expected to have a useful life of only two or three years. Because of the temporary nature of this structure, it was desirable to construct it as economically as possible, consistent with complete safety. But the designers had to keep in mind the fact that although the dam was planned to have a life of only a few years, it might be used for a much longer period.

In April 1958, the Venezuelan firm of OTEHA, C.A., was commissioned to prepare plans and supervise construction of the "low dam," to have a height of 79 ft. Construction started in May, filling of the reservoir started in September, and the dam was completed in December—all in 1958. The water stored during the rainy season of 1958 provided a sufficient additional supply for Caracas to prevent serious shortage in 1959.

The brief time available for design and construction, and the imminence of the annual rainy season, prohibited consideration of a conventional earthfill dam.

Because of the very limited time available and the fact that no borrowpit explorations or laboratory studies had been made, the engineers concentrated immediately on a rockfill design. Alternate plans were prepared for rockfill dams having upstream impervious membranes of reinforced concrete and plate steel. The contractor made cost estimates for each and concluded that the plate steel was the cheaper.

A section through the dam, as constructed, is shown in Fig. 1. The foundation layer is made up of a dense stratum of river gravel. Underlying the gravel is a layer of sandy silt. Time was not available to remove the alluvium down to bedrock. Time was not even available for a careful field and laboratory exploration of the characteristics of the silt layer. A few small-diameter Shelby-tube samples were obtained from various points in the silt layer, and some classification tests and unconfined compression tests were performed.

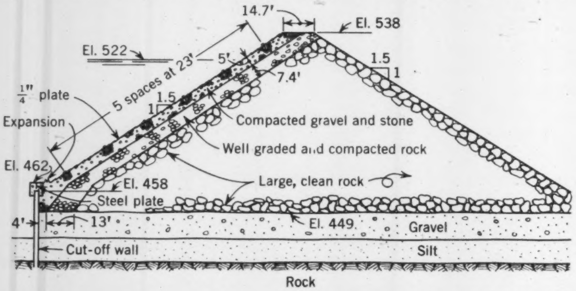

FIG. 1.—CROSS SECTION THROUGH RIO LAGARTIJO DAM SHOWS TYPES OF FILL AND STEEL-PLATE FACING ON UPSTREAM FACE

Stability calculations, based on the results of the unconfined compression tests, indicated that the embankment could be constructed with the slopes and height indicated without danger of foundation shear failure. It was expected that considerable foundation settlement would take place during and after construction. However, since the dam was sufficiently flexible to withstand large differential settlements without rupture, the designers were not particularly concerned about this.

A cutoff wall or seepage barrier, to prevent excessive percolation through the foundation, was provided by constructing a 20-in.-wide concrete wall at the upstream toe. This cutoff wall was constructed according to the patented ICOS process. The trench was excavated to bedrock by drilling holes and excavating between them with a clamshell bucket. During the excavation the trench was kept filled with a bentonite slurry, which prevented the walls from caving and eliminated the need for sheathing. After the trench was excavated to bedrock, tremie methods were used to place the concrete, which displaced the bentonite slurry mixture. Vertical pipes were placed in the trench for subsequent grouting of the bottom of the trench, and to permit drilling into the bedrock for further foundation grouting.

Rock used for the embankment was a hard, sound, bulky limestone, with a maximum size of 10 tons. Only clean, large, hard rock was placed in the embankment, which was constructed in layers ranging from 10 ft to 13 ft. It was sluiced continuously with nozzles of the largest size that could be handled by hand. The sluicing was used to consolidate the embankment and to wash the fines into the interstices of the rock embankment. The 7-ft-thick layer of well-graded fine rock on the upstream slope was obtained as quarry-run fines. Gravel for the 5-ft-thick layer came from deposits in the stream bed. Both of these sloping layers were placed after the main embankment was completed. They formed a cushion to prevent high concentration of stress between the upstream steel face and the large individual rocks in the embankment.

Details of the steel-plate connections are shown in Fig. 2. To provide flexibility in the steel membrane, which would allow it to deform without rupturing or developing extensive tensile stresses, transverse expansion joints were provided vertically up the face on 33-ft centers. An expansion joint consists of a simple fold in a steel plate, which permits considerable movement of the steel at the joint. All joints were continuously welded.

The steel facing is anchored into the underlying gravel layer by bolts embedded in concrete, also shown in Fig. 2. These are spaced 33 ft apart horizontally and 23 ft parallel to the slope.

At the upstream toe of the dam, the steel plate is connected to the top of the concrete cutoff wall, using a double-expansion joint. At the abutments the steel plate is sealed into a 20-in.-wide concrete wall extending 3 ft to 5 ft into the rock.

The steel plates, connections and expansion joints were fabricated in Caracas. Large sections of the upstream face were connected with field bolts before completion of the continuous fillet welds united all the plates and connections into a watertight unit. Oddshaped plates to fit the natural contours of the abutment were cut and welded in the field. The steel plates were placed directly on top of the gravel layer, which had been carefully compacted to a plane sloping surface on grade. In the middle of the day the plates became so hot that they were difficult to handle. Work on them was done only during the morning and evening hours, between 5 and 10 a.m. and 4 and 10 p.m. Connecting the plates when they were hot and expanded was thus avoided.

This construction procedure differs from that on several dams in the United States, where steel plate was laid on gravel. During construction of the United States dams, the steel plates were placed on carefully leveled lightweight steel angles. After the plates were fastened to the angles, the space between the plates and the embankment was backfilled with gravel. The angle frames were left embedded in the backfill, the legs acting as anchors for the plates. This method resulted in a slightly smoother and more attractive face than was obtained by placing the plates directly on gravel. About eight weeks were required for completing the construction of the steel face for the Rio Lagartijo Dam (Fig. 3).

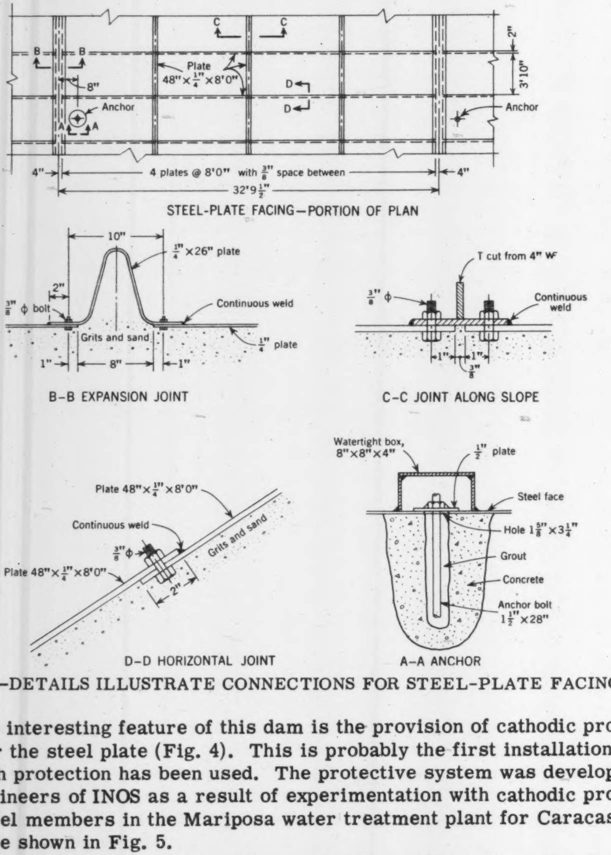
B
B
C
C
2"
3'10"
Plate 48"×1/4"×8'0"
8"
Anchor
A
A
D
D
Anchor
4"
4 plates @ 8'0" with 3/8" space between
4"
32'9 1/2"
STEEL-PLATE FACING—PORTION OF PLAN
10"
1/4"×26" plate
2"
3/8" ϕ bolt
Continuous weld
Grits and sand
1/4" plate
1"
8"
1"
B-B EXPANSION JOINT
T cut from 4" WF
3/8" ϕ
Continuous weld
1"
1"
3/8"
C-C JOINT ALONG SLOPE
Watertight box, 8"×8"×4"
1/2" plate
Steel face
Hole 1 5/8"×3 1/4"
Grout
Concrete
Anchor bolt 1 1/2"×28"
Plate 48"×1/4"×8'0"
Continuous weld
3/8" ϕ
Plate 48"×1/4"×8'0"
Grits and sand
2"
D-D HORIZONTAL JOINT
A-A ANCHOR

FIG. 2.—DETAILS ILLUSTRATE CONNECTIONS FOR STEEL-PLATE FACING

A most interesting feature of this dam is the provision of cathodic protection for the steel plate (Fig. 4). This is probably the first installation on which such protection has been used. The protective system was developed by the engineers of INOS as a result of experimentation with cathodic protection of steel members in the Mariposa water treatment plant for Caracas. Details are shown in Fig. 5.

The cathodic protection system is operated from 110-v alternating current that is fed into a rectifier. The rectifier is grounded directly on the steel plate with a copper cable near the crest of the dam. The positive lead is through shunts to used steel rails that run vertically down the slope of the dam. The steel rails are separated from the steel face by 2-in. rubber insulators. The shunts provide each steel rail with a potential of about 1 v.

Using this system, the upstream steel face of the dam becomes the cathode and the expendable steel rails the anode. Computations indicate that the steel rails will corrode at a rate of about 100 kg per year.

Experiments made by the corrosion engineers of INOS indicate that more current will flow in the system during the first few months after the cathodic protection system is installed than later. Carbonates will deposit on the steel, providing a thin protective covering and a deterrent to the flow of electricity. Consequently the power necessary for cathodic protection will diminish with time.

Performance Record.—The reservoir was almost completely filled by the end of the 1958 rainy season. The level receded as water was drawn off for the Caracas water supply during the 1959 dry season. The reservoir is again filling during the 1959 wet season.

The dam is in excellent shape. No measurements of embankment settlement have been taken, but settlement is not so great as to be noticeable on the crest of the upstream face. Even though the steel-plate facing was not painted, and the cathodic protection was not installed until July 1959, rusting of the steel plates during the first year was negligible (Fig. 6). Considerable leakage appeared downstream from the dam. Through the use of dyes and observation by a diver, it was established conclusively that the leakage was through the upper part of the foundation cutoff wall. The steel face was intact and watertight.

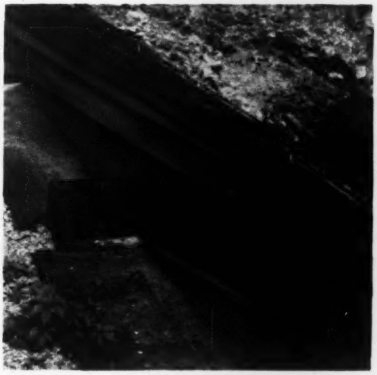

FIG. 3.—STEEL PLATE TERMINATES AT CREST, NOTE EXPANSION JOINT AND RIDGE MADE BY 4-IN. WIDE FLANGE SPLIT INTO A T

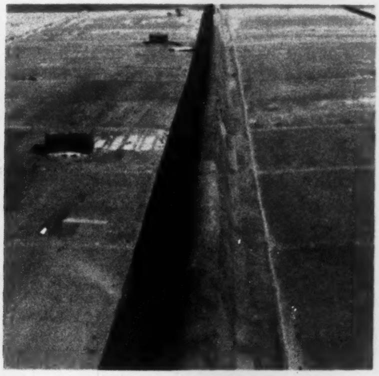

FIG. 4.—STEEL-PLATE FACING FROM CREST, CATHODIC PROTECTION BEING INSTALLED

The Rio Lagartijo Low Dam, described above, was designed and the construction supervised by the Caracas engineering firm of OTEHA, C.A. Work was carried out under the direction of Dr. Diego Ferrer-Fernandez, M. ASCE. Construction was done by the firm of Technica Constructora C. A. of Caracas.

STEEL PLATE ON U.S. DAMS

While there have been very few dams constructed in the United States with steel-plate upstream faces, experience with these dams has generally been good. Corrosion of the steel plate has not resulted in severe maintenance problems.

Rockfill Skagway Dam was built about 1900 in the Rocky Mountains near Victor, Colo. It has a height of 75 ft. The steel plate on the upstream face has a thickness of 1/2 in. at the bottom, 3/8 in. at mid-height, and 1/2 in. at the top. The facing was "riveted and calked in the same thorough manner as in boiler practice."

In the past 30 years the steel face has been given two complete treatments of cleaning, chipping, buffing and painting. The last of these was in 1940. Concerning its present condition, the owner states:

> "Through the years the steel face has become pitted, but not to the extent to become alarming . . . . We are not able to examine the fill side of the steel, but have examined the portion extending above the fill on the cap of the dam at ground level and find that it has suffered less damage than the water side. . . . The steel face has not received the maintenance it should have but . . . it has stood the years very well."

When it is considered that this steel facing is about 60 years old and that the quality of the steel is probably much less uniform and desirable than that in use today, this is a very good recommendation for the durability of steel plate on the upstream face of a dam.

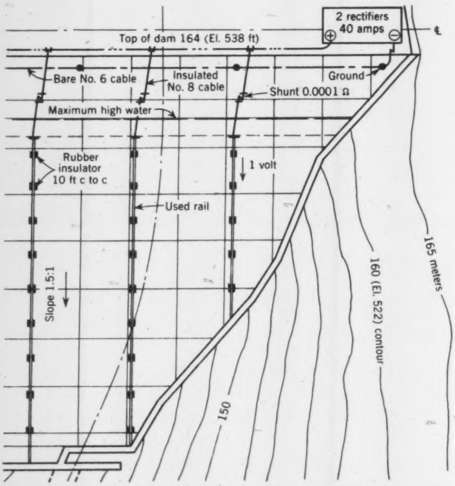

FIG. 5.—CATHODIC-PROTECTION FEATURES ARE INDICATED ON END OF DAM, SEEN IN PLAN. USED RAILS ACT AS ANODES; THE STEEL-PLATE FACING ITSELF SERVES AS CATHODE

Gravel-filled El Vado Dam, with 1/4-in. steel plate on the upstream face, was constructed in New Mexico in 1934. The dam has a maximum height of 175 ft and a 1.5 to 1 upstream slope. The maintenance forces report that "the steel plate is in excellent condition at the present time and with minor inexpensive repair work from time to time should have a life as long as the dam."

Two other earth or gravel dams 80 to 100 ft in height, constructed in the Rocky Mountains in Colorado in the early 1930's, have 1/4-in. steel plate on the upstream face. Detailed information on the maintenance of these dams is lacking but the writer has inspected their steel faces several times in the past 10 years. They appear to be in excellent shape and there are no signs of undue corrosion. From experience with these structures, it can be concluded that corrosion has not been a serious problem on steel-faced dams and that, with reasonable maintenance, the steel face can be considered permanent.

ADVANTAGES OF STEEL-PLATE FACING

Flexibility, watertightness and best distribution of reservoir water pressure are advantages attained by steel-plate facing. The steel plate itself is very flexible and can withstand a great deal of embankment movement without danger of rupture. Through the use of simple and economical expansion joints, of the type used at Rio Lagartijo Dam, the steel-plate facing can be made sufficiently flexible to withstand whatever settlements an embankment or foundation is likely to undergo.

For all practical purposes, steel-plate facing is completely impermeable. When supported on a uniform layer of compacted gravel, the steel plate is not required to resist bending or tensile stresses and acts only as a water barrier or impervious membrane. Used in this fashion, a thin steel plate is capable of acting as an impervious membrane for a reservoir of any depth.

The further upstream the water pressure is resisted by the dam, the greater the stability of an earth or rockfill dam when the reservoir is full. The steel upstream membrane resists pressure on the plane of the upstream face, which is the optimum condition. Assuming proper drainage, there is no problem of decreased slope stability caused by internal pore pressures, following "rapid drawdown" or in the "steady state seepage condition." The dam is not subject to the weakening influence of water seeping through the embankment. Consequently earth dams with steel faces are very different structures from ordinary earth dams and can be constructed on steeper slopes.

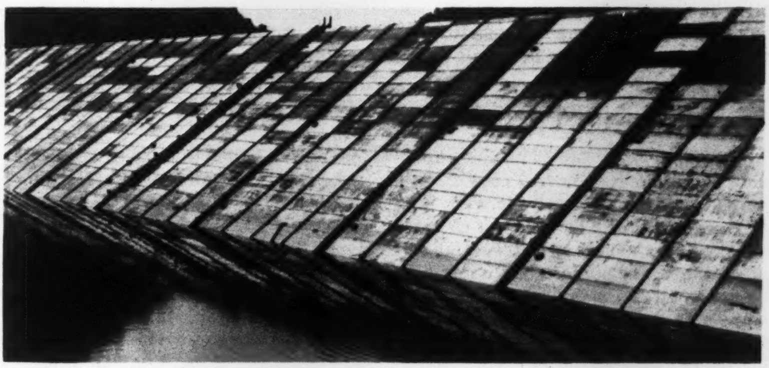

FIG. 6.—UPSTREAM FACE OF RIO LAGARTIJO DAM AFTER ONE SEASON OF USE

Steel-plate facing provides complete protection against the erosive forces of waves.

DRAWBACKS

There are three principal disadvantages in the use of steel plate: (1) the cost, (2) the possibility of damage due to corrosion, and (3) the necessity for drawing down the reservoir level occasionally for inspection and possible repairs. The cost is a major disadvantage, particularly for low dams. At the present time in the United States, it should be possible to put a 1/4-in. steel plate in place on the upstream face of a dam for $20 to $25 per sq yd. For a low dam the cost of the steel plate would be an appreciable part of the total cost of the structure. For high dams, which are thick at the base, the cost of the steel plate in relation to the cost of the embankment becomes much less. Consequently, the higher the dam, the better the competitive position of a steel facing.

Fear of corrosion has probably been the main deterrent to the use of steel plate on dams. However, experience has shown, as discussed above, that the few steel-faced dams in the United States have been remarkably resistant to corrosion. Advances in corrosion engineering should make it possible to reduce this danger to zero, although many engineers would consider this unnecessary.

It would probably be wise to use a steel face on the upstream slope of a dam only where it would be possible, without excessive cost and inconvenience, to completely drain the reservoir at intervals to inspect the condition of the face and repair the steel plate if necessary. On the other hand, any earth or rockfill dam may develop trouble of sufficient magnitude to make it extremely desirable to drain the reservoir completely. Also, some repair work can be done on the steel face by divers, which is an advantage not enjoyed by earth dams with internal clay cores that may develop cracks or leaks. Therefore the necessity for drawing down the water level constitutes only a slight disadvantage for a steel-plate face.

In summary, the writer believes that the data available indicate that steel-plate facing should receive more consideration from dam designers than it has in the past. Some site conditions that make it especially desirable to consider a steel upstream face will now be discussed.

CONDITIONS FAVORING STEEL FACING

No Impervious Soil.—Where impervious soil for an earth core is not available within an economical haul distance, an upstream diaphragm of steel plate may serve the purpose very well.

Possibility of Embankment Cracking.—Recently some earth dams have cracked as a result of differential settlement. This is particularly alarming since there are no reliable criteria by which the possibility of cracking can be analyzed and steps taken to avoid it. The problem is becoming increasingly severe as the height of dams becomes greater and greater. In some dams, intricate and expensive drains and graded filters have been provided to minimize the danger of seepage through possible embankment cracks. The use of an impervious membrane of steel plate would eliminate all danger of failure caused by excessive seepage through embankment cracks.

Economy with Hard Rock.—Where the abutments and foundation for a dam consist of strong rock, and where an ample supply of strong granular embankment material is available, the steel-plate upstream membrane may result in the most economical design, even if clay is available for an impervious core. If a clay core is used, the embankment will probably be constructed with a slope of 1 on 2 1/2 upstream and perhaps 1 on 2 downstream. If the embankment is constructed using only the strong granular material available and an upstream steel face, there will be no danger of embankment sliding, with water held completely outside the structure. Consequently the embankment slopes can be made as steep as practicable from the construction standpoint, which may be 1 on 1.2 or steeper. Obviously the steel-face design would have a much smaller embankment volume and might be considerably cheaper.

Severe Wave Action.—Where severe wave action is expected in the reservoir, and rock for a riprap blanket involves an exceptionally long haul, the cost of any type of suitable wave protection may approximate that of steel plate.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3080

ROCKFILL DAMS:
DALLES CLOSURE DAM

By Robert J. Pope[1]

With Discussion by Messrs. F. L. Lawton; and Robert J. Pope

SYNOPSIS

Construction of a closure dam for The Dalles project in 180-foot depth of water with flows up to 200,000 c.f.s. presented a problem solved by the design and construction of a rockfill structure, two-thirds of which was built under water without use of cofferdams.

INTRODUCTION

The Dalles Dam, located 192 river miles above the mouth of the Columbia River and about 3 miles above the City of The Dalles, is a multi-purpose project in the interest of power, navigation and irrigation. The ultimate capacity of generated power will be 1,743,000 kilowatts, slack water navigation is provided for over 20 river miles and minor irrigation and recreation benefits are also obtained. Construction of the project was started in February 1952, the closure dam was completed April 1957. Four of the turbines are currently in operation and completion of 14 turbines which will provide an installed capacity of 1,119,000 kw is scheduled for November 1, 1960. Normal pool elevation is 160 and average tailwater elevation 74, furnishing a power head of about 86 feet.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1738. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The aerial view of figure 1 shows the site for The Dalles Dam as it appeared in 1951 before start of construction. Bedrock at the site was formed as a series of superimposed basalt flows which are exposed as benches and cliffs on both sides of the river. The most significant structural feature is the Big Eddy fault which crosses the axis of the closure dam in the river channel and undoubtedly is responsible for orientation of the channel. The river flow is deflected from west to south at Big Eddy through a restricted outlet, then turns back west along a narrow channel for five thousand feet, then makes a second S curve to resume its westerly course.

A study of the site logically led to the location of the spillway designed to pass over 2 million c.f.s. and the powerhouse for ultimate development of 1-3/4 million kilowatts on the rock shelf which was skirted by the river channel on three sides. Plenty of room was available at this location for construction with a minimum of cofferdaming while the river continued to flow in its main channel. An aerial photo, figure 2, taken in March 1956 shows the construction as of that date. The navigation lock, spillway, non-overflow dam and powerhouse structures are essentially complete. The only missing major structure for the project to be a closure dam at the narrow outlet from Big Eddy.

General Plan

At the location selected for the closure dam, the river was restricted to a width of about 600 feet, the narrowest point in the vicinity. The original depth of the river in this area varied from 120 to 180 feet. Approximately one million cubic yards of rock from preliminary powerhouse excavation had already been placed in the closure area, reducing the depth of water at the axis to about 55 feet. Upstream the river bottom dips down into the depths of the Big Eddy pool which was 200 feet in depth with a bottom elevation 118 feet below mean sea level.

A rock and gravel embankment was the obvious choice for the type of structure at this location due to an abundant supply of these materials adjacent to the site. However, conventional construction methods, including cofferdams for dewatering the foundation presented difficulties which were readily apparent.

The general plan developed for constructing the closure dam consisted of first constructing a dumped rock diversion fill to divert the river flow through eight skeleton units in the powerhouse, then completing the closure dam by raising the dumped rock fill and placing an impervious blanket on the upstream face.

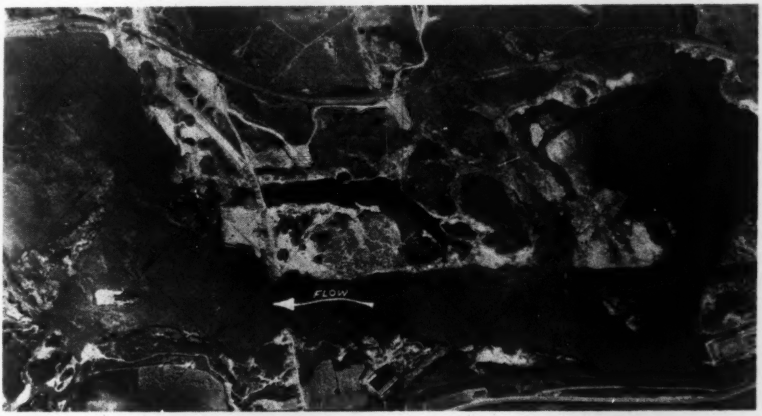

FIG. 1.—VIEW OF DAM SITE, APRIL 1951

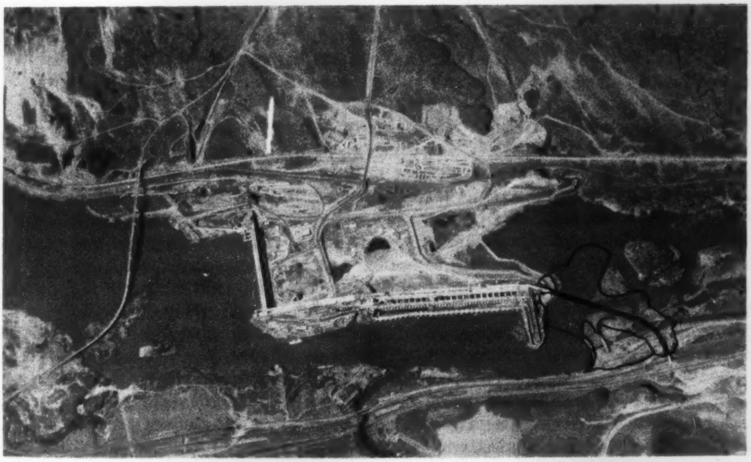

FIG. 2.—VIEW OF CONSTRUCTION, MARCH 1956

Schedule Limitations

The most difficult aspect of design of The Dalles closure dam was development of a plan which could be scheduled for completion within rigid time limitations imposed by the duration of low water season for diversion through the powerhouse, weather conditions and a minimum of interference with river navigation and fish migration.

The favorable low water season for construction on the Columbia River extends from October through February during which time the mean daily flow is approximately 100,000 cubic feet per second. Flows greater than 200,000 cubic feet per second have not occurred prior to November 7, and flows greater than 300,000 cubic feet per second have not occurred prior to December 23. All winter floods are of short duration. The longer duration spring freshets due to snow melt, starts its rise after April 1, generally peaks during June and has produced peak flows as high as 1,240,000 cubic feet per second. It was, therefore, necessary to design the diversion fill so that it could be constructed during the early winter months, during possible winter flood flows up to 300,000 cubic feet per second. It was also necessary to have the closure dam sufficiently complete to close the gates to the skeleton units of the powerhouse and raise the pool to pass the flow over the spillway prior to the spring freshet, as flood flows in excess of 300,000 c.f.s. through the powerhouse would threaten serious damage. This was because the structure could not be designed for summer floods.

Since barge traffic on the river is an important factor in the regional economy, it was necessary to hold to a minimum the time of closure to this traffic. Before construction of The Dalles Dam, navigation of the river for about 12 miles of non-navigable river had been bypassed through The Dalles-Celilo canal with its entrance just downstream of the closure damsite. The actual time of closure to barge traffic was less than 3 months. This was accomplished by permitting continued operation of The Dalles-Celilo Canal until necessary to close the gap for completion of the dam. A transfer pipeline was also provided from below the dam to the canal above to permit continued movement of petroleum products during the short period of navigation interruption.

A more-or-less continuous migration of salmon and steelhead up the Columbia River to spawning beds above the dam site occurs from late April through October. Since obstruction of these runs would result in considerable economical and recreational loss to the region, obstruction of the river channel had to be delayed until October and the pool raised by the following April to enable operation of the fish passing facilities. This would avoid the expense and difficult operations of temporary fish passage facilities.

Fortunately the favorable construction season with regard to river flows, navigation and fish migration coincided reasonably well. However, these restrictions imposed a 6-month schedule for the placement of almost 2,000,000 cubic yards of material in the main part of the closure dam during the winter season with severe winter weather and high river flows probable during the last 4 months. It was therefore necessary to facilitate construction in every possible way. Construction procedures were planned so that occurrence of normal winter floods or icing conditions would cause only minor interference with the schedule and emergency measures were planned to assure sufficient completion to allow closure of the powerhouse diversion even in the event of the occurrence of a record winter flood.

Diversion Scheme

Completion within the required time was dependent upon development of a diversion method which could be rapidly completed with certainty under all expected river and weather conditions. Previous river diversions of the Columbia and Pend Orielle Rivers had been end dumped using rock varying in maximum weight from 5 to 35 tons, or constructed with tetrahedons placed from overhead cableways. The use of heavy rock or similar construction methods were considered necessary during early planning for The Dalles diversion fill. However, further study indicated considerable advantage could be obtained by the construction of an adequate diversion channel to the skeleton units of the powerhouse. Excavation of a diversion channel 695 feet in width with a bottom elevation of 60 reduced the head required for diversion and actually permitted 25 percent of the river to flow through the 8 skeleton units of the powerhouse before the diversion fill was started. An increased width of diversion fill was also advantageous in that it reduced the gradient through the closure gap and in addition, provided adequate working width for the contractors operations on the fill. An increased width of diversion fill was economically feasable because the diversion fill was to be incorporated into the completed closure dam. Construction of a fill approximately 250 feet in width with an upstream leading edge created more favorable hydraulic conditions, by dissipating most of the energy at the upstream edge which permitted the smaller rock to be deposited downstream but within the width of diversion fill.

Because of the limited time available for construction of the diversion and unusual channel and hydraulic conditions, a series of model studies were conducted at the Bonneville Hydraulic Laboratory. The studies were made in a 1:40 scale model of the closure area. A number of variations in the construction method were studied including various rock sizes and different placement procedures. These studies varified the feasibility of constructing the diversion fill by end-dumping methods and demonstrated that a closure could be made using quarry-run rock having a maximum size of 1,000 pounds.

Blanket Design

Design of the upstream blanket sections was governed by the usual criteria regarding slope and foundation stability, seepage control and filter protection. Since construction time was of the essence, complicating refinements in section were avoided to the fullest possible extent and serious consideration was given to the practical limitations inherent in underwater work with respect to segregation of materials during placement, the difficulty of inspection, and the probable accuracy of underwater survey measurements.

Extensive explorations were conducted in search for materials suitable for blanketing the upstream face of the rockfill dam. These explorations revealed adequate supplies of poorly graded, sandy gravels; well graded, sandy gravels; and an abundance of non-plastic silts and fine sands. Silts and fine sands were considered undesirable for underwater placement because of flat slopes required and sensitivity to flow slides. Design of the blanket section and selection of materials was dependent on first determining the method for underwater placement of materials. Serious consideration was given to devising methods for placement which would minimize segregation and insure stable construction slopes. Most of the methods considered had disadvantages of slow construction rates, high costs, or required special equipment which would be expensive, untested and unfamiliar to the contractors. The method considered most feasible was to place all underwater material by end-dumping at the water surface or slightly above except for a minimum width of upstream blanket which would be lowered in place. All underwater slopes were built on the natural angle of repose of the dumped materials except the final outside slopes of the completed dam. This method limited the choice of blanket materials to granular materials with non-plastic fines which would assume reasonably steep underwater slopes without incurring danger of large slump failures during construction. End dumping and building slopes on the natural underwater angle of repose had the advantage of low cost, fast placement rate and required a minimum of underwater survey control.

A typical cross section of the Closure Dam is shown on figure 6. The top of the dam is 295 feet above the upstream toe of the blanket material and the length of dam approximately 2,000 feet. The upstream slope is 1 on 2 above elevation 90 and 1 on 2.3 below elevation 90, the major portion of which was constructed underwater. The downstream slope is 1 on 1.5. Top width of the dam is 30 feet to accommodate road and railroad access to the upper deck of the powerhouse. A berm on the downstream slope provides access to the lower deck of the powerhouse. Base width is approximately 1,200 feet. The section below elevation 90, approximately 75 percent of the cross sectional area in the river channel, was designed to obtain the greatest possible advantage of the rapid placing rates made possible by end dumping methods.

Materials of the various zones grade in coarseness from quarry-run rock to sandy gravel. Each zone acts as a filter for the next adjacent upstream zone of finer material. Rock from preliminary powerhouse and cofferdam excavations which was placed in the closure area to approximately elevation 20 during the early stages of construction was dumped by truck from a barge anchored in the river for a pontoon roadway. The maximum depth of this fill was about 125 feet which left 55 feet depth of water over the fill at low river flow. The remaining quarry run rock and spalls for the embankment came from a quarry on the Oregon shore near the end of the abutment. The rock was a hard, durable basalt with a unit weight of 175 to 180 pounds per cubic foot. The rock was used in quarry run sizes which ranged from 200 to 300 pound average weight. Rock spalls were reasonably well graded with a 12-inch maximum size. Rockfill and spalls placed above the water surface were spread in layers 3 to 5 feet in thickness depending on the maximum size of rock and compacted with the spreading and hauling equipment.

Gravel was obtained from a river bar deposit about 3 miles upstream of the dam. Material from this source consisted of skip graded sandy gravel with a maximum size of about 4-inches and lacking material between the No. 4 and 30 mesh screen size. The gravel was used as it came from the pit, placed below water by end dumping off the edge of the fill and above water, spread in layers 18-inches thick, and compacted with 2 passes of a 50-ton rubber tired roller.

The upstream blanket was constructed of sandy gravel obtained from a high terrace deposit about 3 miles east of the site. This material was a well graded sandy gravel containing about 35 to 50 percent sand. The blanket materials were placed in the same way as the gravel except that the outer wedge below elevation 90 was lowered into place so as to prevent segregation and thereby insure that the coarse materials which accumulated at the base of the dumped material would be adequately blanketed.

Providing stability against sliding of a dam constructed entirely of granular materials in the manner described is a relatively simple problem. A safety factor of 1.0 against surface slides is inherent in the method of underwater construction of slopes on the natural angle of repose. Flattening the finished upstream and downstream slopes increased the minimum safety factor to 1.5 or better for any applicable condition of analysis. The safety factor of the upstream slope is further increased with a normal reservoir level due to the sloping upstream impervious blanket.

Laboratory permeability tests on the sandy gravel blanket material indicated a permeability ratio of from 12 to 20 x 10^{-3} feet per minute. This is many times faster than the permeability usually considered necessary for an impervious zone, but the computed seepage loss of 100 to 170 c.f.s. through the newly completed blanket was expected to decrease considerably with time as silt is deposited on the outer slope.

Foundation Exploration

Preliminary foundation explorations in the closure channel had been limited in extent due to the difficulties of underwater explorations, obvious exposures of solid basalt rock on each side of the channel and because it was expected that the high river velocities in the channel would keep the river bottom swept clean. This had been partially verified by a few difficult and costly inspections made by a team of divers in 1945. When more detailed information was required for final design, it was found that the river currents were too swift in many areas to obtain a reasonably complete examination of the river bottom by use of divers. A closed circuit television camera in a watertight case operated on cables from a barge was then utilized to survey the bed of the river.

The camera was operated in swift water by first lowering a heavily weighted anchor line to the river bottom. The camera was then lowered with slip rings attached to the anchor line. The camera was mounted in a tripod frame provided with lights for underwater illumination as shown on figure 3. Record photographs of the foundation conditions were made from the TV monitor screen. Riverbed materials were identifiable on the television screen as shown in the photograph of figure 4.

Considerable information was quickly and economically obtained by use of the underwater television camera. It was learned that approximately one-third of the foundation was covered by sand or gravel, and boulder deposits were found at the base of steep underwater cliffs. The upstream toe of the rock fill placed during powerhouse excavation was carefully mapped. Annual surveys by both TV camera and hydrographic surveys were conducted following each spring freshet to observe if any erosion of the fill had occured and the amount and nature of any deposition in front of the fill. The rock fill withstood four annual flood flows before completion of the closure without appreciable erosion and a maximum of 7 feet of coarse gravel was deposited adjacent to the upstream toe of fill. Underwater mapping with the TV camera eliminated areas of exposed bedrock from further exploration and difficult and costly drilling was restricted to gravel and sand areas. The next step was to determine the depth, gradation and permeability of the alluvial material. Considerable difficulty was experienced in obtaining suitable borings due to swift river currents and water depths of 100 to 120 feet. Drilling was accomplished by

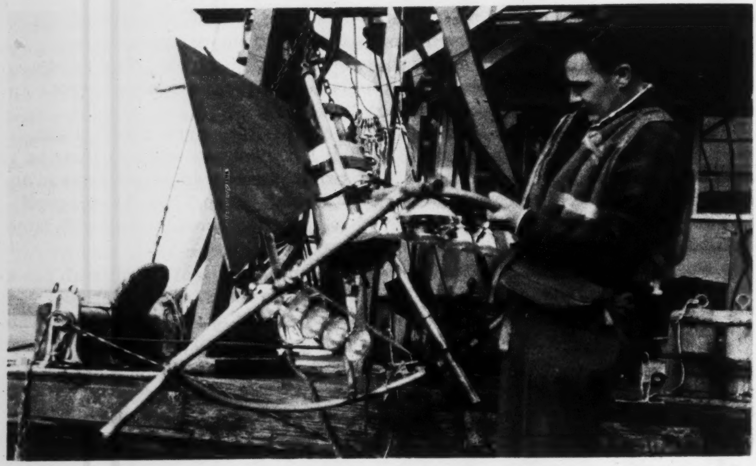

FIG. 3.—TV CAMERA FOR UNDERWATER USE

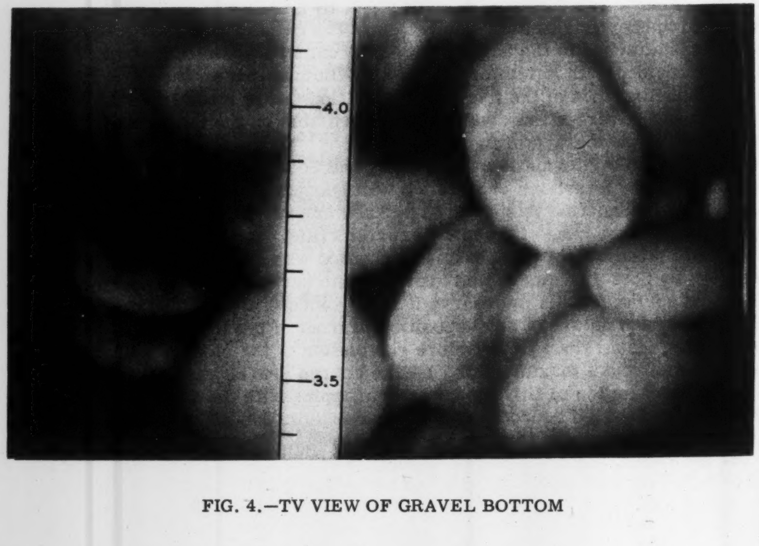

FIG. 4.—TV VIEW OF GRAVEL BOTTOM

operating through the center well of a barge. An outer 6-inch casing, guyed near the bottom and midpoint, was lowered and seated into the riverbed. A 4-inch casing was lowered inside this and the boring and inside casing advanced to bedrock by the usual methods of chopping, washing and driving. Bedrock was cored with a diamond bit.

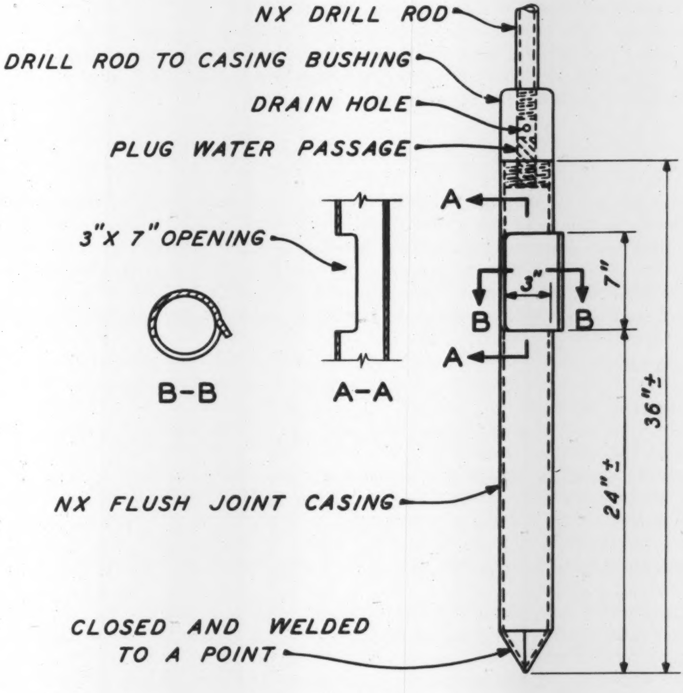
NX DRILL ROD
DRILL ROD TO CASING BUSHING
DRAIN HOLE
PLUG WATER PASSAGE
3"X 7" OPENING
A
A
B
B
3"
7"
36"±
24"±
B-B
A-A
NX FLUSH JOINT CASING
CLOSED AND WELDED TO A POINT

FIG. 5.—SIDE INTAKE SAMPLER

Samples of overburden sands and gravels were obtained as the 4-inch casing was withdrawn using a side-intake soil sampler which was shop built from a 3-foot section of NX flush joint casing as shown on figure 5. This sampler was lowered on the drill rods to the elevation at which a sample was desired and the 4-inch casing then raised until the bottom was above the opening in the sampler, but not above the top shoulder of the sampler. The sampler was then rotated slightly and withdrawn. A total of 23 borings were made in the river bottom of which 12 were sampled by the above method. In addition to the borings, the top of rock was sounded by probings to define the edges of the fault. Consistent results were obtained by the above sampling method and it is believed that the samples obtained were representative of foundation materials, although the opening in the sampler would not permit the entrance of stones over 3 inches in diameter.

The river channel explorations revealed a gravel filled fault 90 to 130 feet in width and up to 80 feet in depth, extending from approximately the centerline of the dam to the bottom of Big Eddy. A barrier across the fault at approximately the centerline of the dam prevented the sand and gravel filling from being transported downstream. Rock walls of the fault are evidently very steep. One boring located at the edge of the fault broke through rock into underlying gravel which indicated the presence of an overhanging rock wall. The fault is filled with stratified deposits of fine sand, skip-graded sandy gravel and coarse open gravel.

Foundation Studies and Treatment

The amount of seepage and the location and magnitude of critical seepage gradients in the sand and gravel filled fault were studied with the aid of flow nets based on somewhat simplified foundation conditions as found from the river borings. For this purpose the gravel filled fault was divided into 4 strata based on average relative permeability ratios. The flow nets were considered only approximations of the actual seepage pattern, but they indicated locations of critical areas and the relative values of gradients and seepage quantities. Permeability ratios used in the flow nets were obtained from laboratory permeability tests performed on samples obtained in the side-intake sampler.

The flow nets drawn to represent seepage conditions in the gravel and sand beneath the closure dam without any foundation treatment such as the blanket extension shown on figure 6, indicated seepage gradients sufficient to cause migration of the fine sands in the foundation into the gravel and rock zones. Such piping would not greatly endanger the stability of the closure, but could result in subsidence of the foundation under the blanket and gravel zones sufficient to disturb the blanket and cause concentrated leakage which would be difficult to correct.

Corrective methods studied included an upstream blanket, placement of graded filters to prevent transportation of fines, a pile cut-off to rock and a partial cutoff trench. The plan of blanketing the area upstream from the toe of the dam was adopted as the most practical and economical for underwater construction. The blanket is an extension of the lowered-in-place portion of the blanket zone, and constructed using the same well graded sandy gravel and placed by the same method. The blanket extends upstream from the toe of the dam for a distance of approximately 250 feet. The seepage gradients under the blanket and gravel zones of the dam were effectively reduced, and the estimated quantity of seepage through the foundation was reduced from 260 to 110 cubic feet per second.

Construction

A contract for construction of the closure dam was awarded in October 1955. This allowed nearly a year for construction of haul roads to the borrow areas and quarry site and for construction of a portion of the dam at the left abutment. The diversion fill was started September 10, 1956 by dumping rock

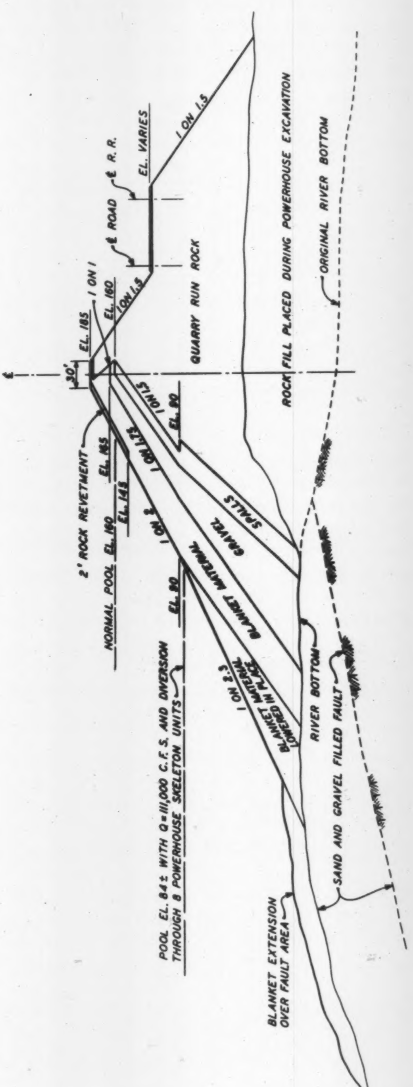
℄
30'
EL. 185
1 ON 1
EL. 160
1 ON 1.5
℄ ROAD
℄ R.R.
EL. VARIES
1 ON 1.5
QUARRY RUN ROCK
ROCK FILL PLACED DURING POWERHOUSE EXCAVATION
ORIGINAL RIVER BOTTOM
2' ROCK REVETMENT
NORMAL POOL EL. 160
EL. 165
EL. 145
1 ON 1.75
1 ON 1.75
EL. 90
1 ON 2
SPALLS
GRAVEL
BLANKET MATERIAL
EL. 90
POOL EL. 84± WITH Q=111,000 C.F.S. AND DIVERSION THROUGH 8 POWERHOUSE SKELETON UNITS
1 ON 2.3
BLANKET MATERIAL LOWERED IN PLACE
RIVER BOTTOM
SAND AND GRAVEL FILLED FAULT
BLANKET EXTENSION OVER FAULT AREA

FIG. 6.—TYPICAL CROSS SECTION

in the main river channel from the Oregon shore. The actual diversion closely followed the method which had been verified by model study. A 240-foot frontal width of fill provided adequate space for an expedited rock placement program. The upstream edge of the fill was maintained slightly in the lead to minimize loss of rock outside the dam section as the model study had demonstrated. The final 25-foot width of river channel was closed in a period of 90 minutes on October 17, 1956 by concentrating all dumping at a 60-foot wide leading edge. During this critical stage of closure the river flow was 111,000 c.f.s. and a head of approximately 4 feet through the closure gap created velocities of about 12 feet per second. The photograph of figure 7 shows the diversion fill nearing completion.

End dumping of spalls and gravel followed advancement of the diversion fill as closely as river velocities through the remaining gap would permit. All material to be dumped under water was first dumped on the edge of the fill and then dozed over the shoulder which was maintained not more than 5 feet above the water surface. These precautions assured under water slopes on the natural angle of repose. No difficulty was experienced with shoulders breaking back or slides within placed material. The under water dumped angle of repose of the gravel averaged 1 on 1.8, blanket material 1 on 1.7, and rock or spalls 1 on 1.3. Material for the lowered in place blanket and upstream blanket extension was placed by use of 12-cubic yard skips lowered with a 100-ton derrick barge. The skips were not dumped until in contact with the bottom in order to minimize segregation during placement.

Under water inspections were conducted by divers at the completion of each stage of blanketing. Particular attention was given to all areas at which the blankets contacted steep under water slopes at the sides of the deep channel and adjacent to the Oregon and Washington shores. These inspections revealed unfilled rock overhangs and unblanketed talus slopes at the base of under water cliffs. Gravel was packed into the rock overhangs by jetting and additional blanket material was placed adjacent to the contact areas in the form of fillets to assure adequate sealing at these critical zones.

Under water surveys prior to start of construction were conducted mainly with sonic depth finders, but construction surveys by lead line for greater accuracy were feasible due to the existence of quiet water after diversion had been accomplished.

The closure dam was essentially complete in time for the scheduled filling of the reservoir. The pool was raised in two increments. On March 10, 1957 at 10:00 AM the 8 intake gates in the diversion powerhouse units were simultaneously closed, and in 4-1/2 hours the pool had raised approximately 40 feet to spillway elevation 121. On the following weekend, March 15 and 16, the spillway gates were lowered and the pool raised to elevation 155. Navigation was then restored through the new navigation lock and the fish passage facilities placed in operation in time for the spring salmon run. A full pool elevation of 160 was delayed until July 17 in order to complete necessary revisions on an upstream railroad bridge. Figure 8 shows the closure dam essentially complete with the pool at elevation 155.

Performance Record

The only observable leakage since completion occurred on the Oregon shore between the left abutment and the old ship canal. Leakage of about

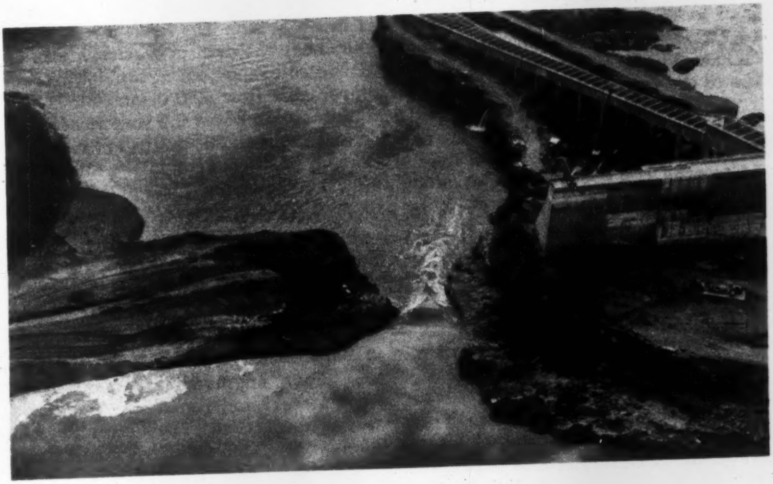

FIG. 7.—DIVERSION FILL, OCTOBER 15, 1956

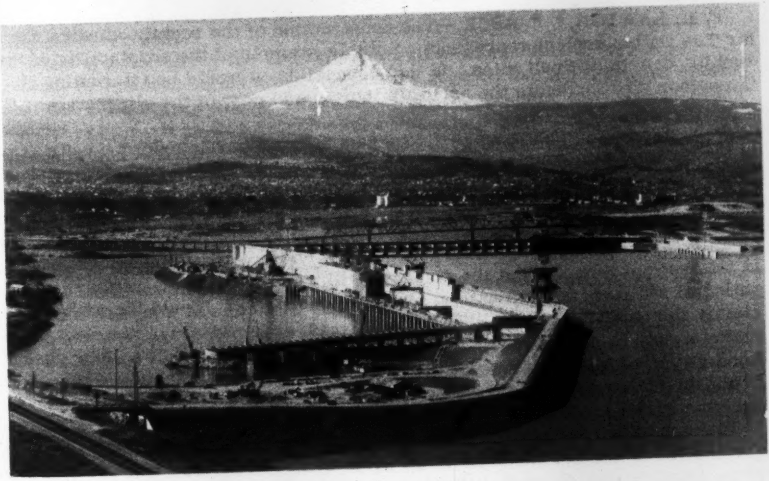

FIG. 8.—COMPLETED CLOSURE DAM, MARCH 26, 1957

100 gallons per minute at the extreme end of the abutment was sealed by grouting joints in the rock which probably had been loosened by railroad construction. Considerable leakage was observed in the old ship canal when the pool was first raised, but no attempt was made to stop this leakage. It is probable that the total leakage through and under the dam is less than was anticipated because no salmon have been attracted to the pool at the downstream toe of the dam as would be the case if appreciable flow still occurred.

Settlement hubs along the top of the dam indicate about 0.5 foot of settlement occurred in a 3-week period during and immediately following raising of the pool. About 1 foot additional settlement has occurred in the following 14 months at a gradually diminishing rate. The settlement has been proportional to the depth of fill as would be expected and has been slightly less than anticipated.

ACKNOWLEDGMENTS

The Dalles closure dam was designed and constructed under the direction of the Portland District, Corps of Engineers, Colonel Jackson Graham, District Engineer; Ben L. Peterson, M. ASCE, Chief, Engineering Division; Paul Thurber, A.M. ASCE, Chief, Foundation and Materials Branch. Atkinson and Ostrander Company was the Contractor for construction. The writer is grateful to Mr. Ben L. Peterson, Chief, Engineering Division for review and construction criticism of this paper.

DISCUSSION

F. L. LAWTON,[2] F. ASCE.—The construction of the rockfill closure dam for The Dalles project represents a notable example of the adaptability of the rockfill dam to difficult sites. In this connection, it would be interesting if the author could indicate the probable increase in cost of the alternative closure.

It is clear from the paper that "the general plan developed for construction of the closure dam consisted of first constructing a dumped rock diversion fill to divert the river flow through eight skeleton units in the powerhouse, then completing the closure dam by raising the dumped rock fill and placing an impervious blanket on the upstream face". However, it is not immediately clear to what elevation the diversion fill was constructed, although the paper is interpreted to mean that the diversion fill was built up to a maximum elevation of 170 feet, approximately, above the foundation. Could the author clarify this point?

Reference is made to the fact the diversion fill withstood ". . . four annual flood flows before completion of the closure without appreciable erosion . . ." Could the author indicate the velocities which are known to have occurred over the surface of the diversion fill?

The use of the underwater television camera for examination of the diversion fill is another instance of the great utility of the camera, particularly when it can be used in such difficult conditions as the Big Eddy Pool. The procedure adopted for the placement of the blanket appears to be a very sound one. Could the author indicate the loss of material which is estimated to have occurred in placement? In connection with permeability of the sandy-gravely blanket, it is indicated ". . . the computed seepage loss of 100 to 170 cfs through the newly completed blanket was expected to decrease considerably with time as silt is deposited on the outer slope". Is there any indication of this proving to be the case?

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[2] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

The paper also makes reference to the use of sonic depth finders for the underwater surveys made prior to the start of construction, with the lead line being used after diversion. Could the author indicate the relative accuracy of the two methods as found under the site conditions?

ROBERT J. POPE.[3]—Alternate sites for The Dalles Project were considered during the early planning stages, but rejected due to unfavorable economics. No feasable alternate method for construction of a closure dam at the selected site was devised due to the depth of river at this location.

The river bottom within the foundation area for the diversion fill was at approximate elevation -85 MSL. Rockfill obtained from excavation for the powerhouse was placed in this area as shown on fig. 6 to approximate elevation +20 during the early stages of construction. This was the portion of the fill which sustained four annual flood flows before completion of the closure without appreciable erosion. River velocities in the closure area were not measured during these floods but model studies[4] indicate the velocities during the 823,000 cubic foot per second maximum flood of 1956 to have been 30 feet per second at 5 foot depth, 32 feet per second at mid depth and 26 feet per second at 5 feet above the bottom.

The fill lift which diverted the river flow through eight skeleton units in the powerhouse was end dumped about 5 feet above the river surface or at approximate elevation 85. The height of the diversion lift was therefore about 65 feet above the surface of previously placed rock, but 170 feet above the original river bed.

Loss of rock during the diversion operation was held to a very small amount by maintaining an upstream leading edge on the 240 foot wide diversion fill. Most of the energy was thereby dissipated near the upstream edge and the smaller rock was deposited downstream but still within the width of the diversion fill. Only the finest rock was carried downstream of the dam section. The gravel and blanket materials were placed in relatively quiet water and no loss of these materials occured.

Unfortunately no method of measuring the quantity of seepage through the embankment or foundation has been devised. Considerable seepage could occur without noticeable evidence due to approximately 180 feet depth of tailwater. Underwater examination of the upstream face for silting since completion of the dam has not been made.

Reasonable agreement was obtained between sonic depth finder and lead line soundings for relatively level bottom conditions. However, when sounding to sloping or irregular surfaces, the accuracy of the sonic method decreased in proportion to the depth of water and slope of the bottom surface.

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[3] Chf., Soils Engr. Section, Portland District Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.

[4] "The Dalles Diversion Made With Rockfill Dam," by Marvin J. Webster, Civil Engineering, February, 1957, pp. 59.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3081

ROCKFILL DAMS: BERSIMIS SLOPING CORE DAMS

By F. W. Patterson[1] and D. H. MacDonald,[2] M. ASCE

With Discussion by Messrs. F. L. Lawton; and F. W. Patterson and D. H. MacDonald

SYNOPSIS

The Bersimis No. 1 hydroelectric power development in Canada involved the construction of two earthfill and two rockfill dams. This paper describes the site, design, construction, and performance of these dams.

INTRODUCTION

The Canadian Shield area of the Province of Quebec offers some of the world's most favourable sites for development of hydro-electric power. These hydroelectric projects usually involve the construction of large dams. In the past many of the major dams were constructed of concrete, but now, because of modern development in dam construction and also for economic reasons, greater consideration is being given to earth and rockfill dams.

The recently completed Bersimis No. 1, previously known as the Bersimis - Lac Cassé Development,[3] involved the construction of two earthfill dams, and two rockfill dams, which are among the first of their type to be built in

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Note.—Published essentially as printed here in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1740. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[3] "Bersimis - Lac Cassé Hydro-Electric Power Development," by E. Rousseau, The Engineering Journal, Vol. 39, pp. 373-386.

this region. It is proposed to describe in this paper the site and the design, construction and performance of these two rockfill dams.

Topography

The topography of a large part of the Province of Quebec to the north of the St. Lawrence River is rugged and is traversed by several large rivers which empty into the St. Lawrence. One of these rivers, the Bersimis or Betsiamites, flows in a southeasterly direction and reaches the St. Lawrence about 320 miles downstream from the city of Montreal. Bersimis No. 1 is located at the outlet of Lac Cassé approximately 85 miles from the mouth of the Bersimis River. The general location is shown on Fig. 1.

Between the outlet of Lac Cassé and the mouth of the river, the Bersimis, in its natural state, fell approximately 1,150 feet. More than 700 feet of this drop occurred in the first 20 miles below Lac Cassé. This topographic feature offered ideal conditions for the development of power on the river. Conditions also were favourable for construction of a dam at the outlet of Lac Cassé to provide additional head for power development.

At the outlet of Lac Cassé the Bersimis enters a relatively narrow valley where a fall of 90 feet occurs. A small tributary, the Desroches River, joins the Bersimis just downstream of the falls. There appeared to be a choice of constructing dams above the falls across both the Bersimis and Desroches valleys or, alternatively, one dam downstream of the falls and below the confluence of the two rivers. The former alternative was chosen. Fig. 2 shows the location of these dams relative to the locations of the other works included in the project.

Geology

General Geology

The Bersimis dams are located near the southerly boundary of the region known as the Canadian Shield which covers almost one-half the surface area of Canada. This region is an uplifted peneplain of small relief and of elevation almost entirely below 2,000 feet. The Canadian Shield consists of Precambrian rocks which were worn down and rounded off by intensive continental glaciations during the Pleistocene Epoch. These several glaciations completely rearranged the existing drainage pattern and covered the area with an assortment of glacial till and glacio-fluvial deposits. Subsequent to the last glaciation a new and disorganized drainage pattern was created and some resorting of the glacial deposits has occurred through fluvial action. Recent uplift of the land mass has resulted in the existence of marine deposits in the areas immediately adjacent to the lower St. Lawrence River.[4,5]

The rocks of the Bersimis area belong to that part of the Canadian Shield known as the Grenville Series. This is a complex of sedimentary gneisses which have been intruded by anorthosites, gabbros, granites and diorites and their gneissic derivatives, and pegmatites. In the Labrieville area the older sedimentary gneiss or paragneiss and the granitic gneisses are underlain by a batholithic formation of anorthosites and gabbros which outcrops at the surface as intrusions of varying size. A considerable amount of faulting and

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4. "Geology and Economic Minerals of Canada," by C. H. Stockwell and Officers of the Geological Survey of Canada, Economic Geology Series No. 1, Geological Survey, Dept. of Mines and Technical Surveys, Ottawa, 4th ed., 1957, 517 pp.
5. "Geology of Quebec--Volume II--Descriptive Geology," by J. A. Dresser and T. C. Denis, Geological Report 20, Dept. of Mines, Province of Quebec, 1944, 544 pp.

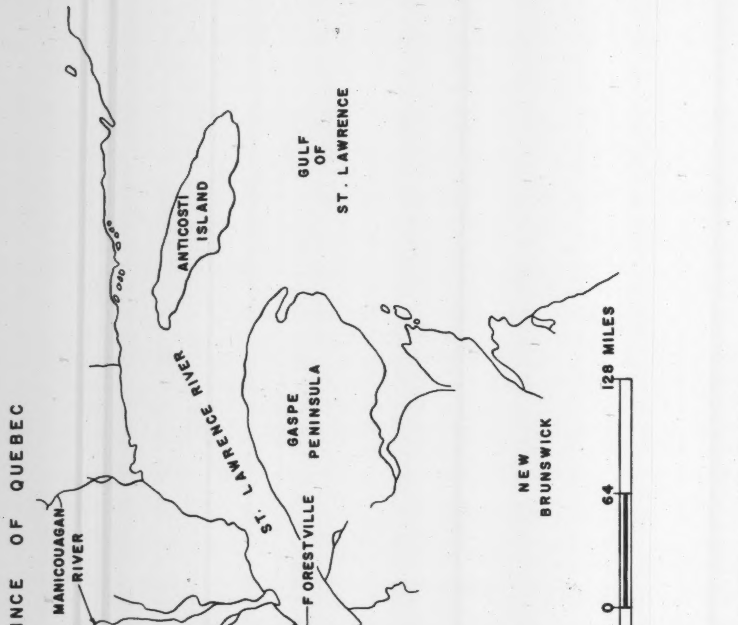
PROVINCE OF QUEBEC
MANICOUAGAN RIVER
OUTARDES RIVER
BERSIMIS No. 1
LAC CASSE
LABRIEVILLE
FORESTVILLE
SAGUENAY RIVER
PERIBONKA RIVER
LAKE ST. JOHN
CHICOUTIMI
QUEBEC CITY
MONTREAL
ST. LAWRENCE RIVER
ANTICOSTI ISLAND
GULF OF ST. LAWRENCE
GASPE PENINSULA
NEW BRUNSWICK
SCALE 64 32 0 64 128 MILES

FIG. 1.—LOCATION OF DEVELOPMENT

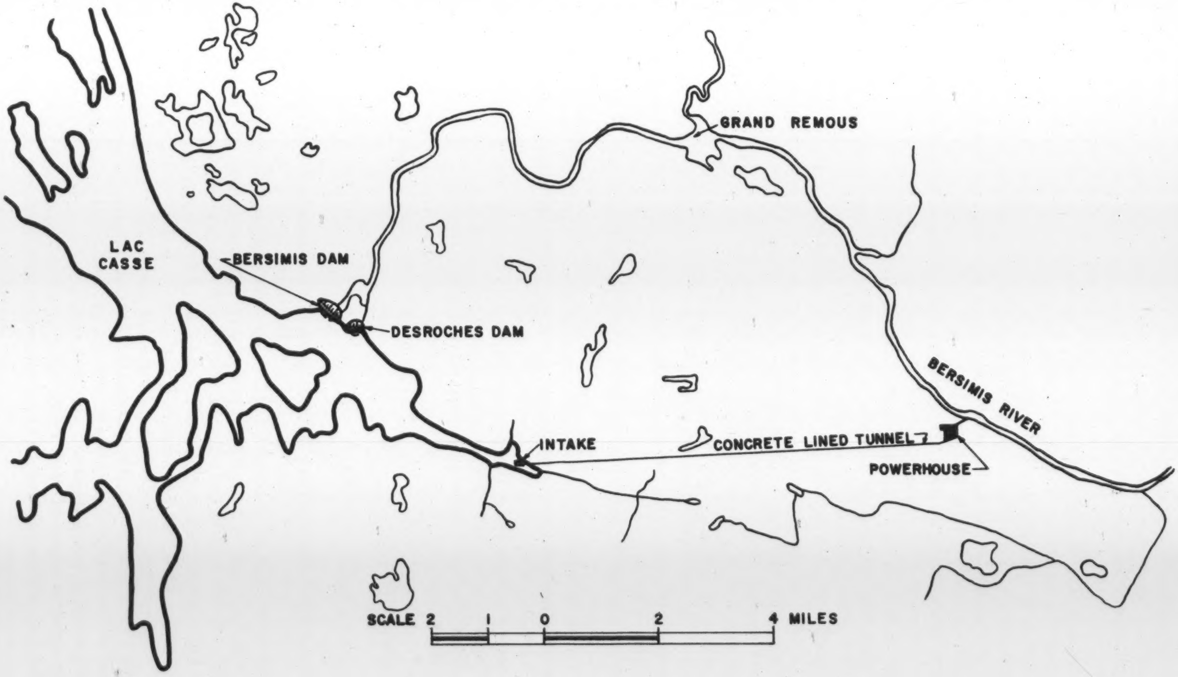
GRAND REMOUS
LAC CASSE
BERSIMIS DAM
DESROCHES DAM
BERSIMIS RIVER
INTAKE
CONCRETE LINED TUNNEL
POWERHOUSE
SCALE 2 1 0 2 4 MILES

FIG. 2.—PLAN OF DEVELOPMENT

shearing has occurred in both Precambrian and Postcambrian times but these fractures are almost invariably well-healed and recemented now. Folding is locally very pronounced in the gneisses and numerous granite, diorite, and pegmatite dykes cut through the rock complex overlying the anorthositic mass.[6]

During the last Pleistocene glaciation, the eastern portion of Canada was covered by the Labrador ice sheet which had its centre located near Lake Kaniapiskau, approximately 250 miles north of Labrieville. Because of this proximity to the glacial centre, the Bersimis area was completely scoured down to bedrock and the soils which now exist are entirely the result of glacial deposition or post-glacial erosional forces. The glacial soils consist of tills which are to be found as side hill deposits, and because of the nearness to the glacier centre and the mineralogy of the bedrock they contain only small percentages of fine-grained or clay-type particles. The most commonly occurring overburden formations are the terrace deposits which are of glacio-fluvial and glacio-lacustrine origin. These terraces consist for the most part of stratified silts and fine sands but also include some coarser sand and gravel layers. They occur in thicknesses ranging up to several hundred feet and cover areas of considerable lateral extent. Through these terraces, and to a much smaller degree through the till deposits, the new drainage system has been cut, and the eroded terrace soils have been redeposited as ill sorted recent river and lake bed formations.

Areas of sensitive marine clays are to be found along the shores of the St. Lawrence River, but these have not been found at greater elevations than approximately 400 feet. Along the Bersimis River these have been noted as far as 45 miles upstream from the mouth, but they are absent in the area of the Bersimis dams.

The Canadian Shield is seismically one of the least active zones known. However, shallow shocks of moderate intensity do occur near the edges of the shield, particularly in the St. Lawrence River valley, and seismic considerations must be included in the design of structures in this region. The nearest earthquake of sizable magnitude occurred in 1925 about 70 miles downstream from Quebec City and approximately 125 miles from Labrieville, and its recorded intensity on the Gutenberg-Richter scale was 7. In 1935 a tremor of intensity 6.2 occurred in the Temiskaming area, approximately 500 miles from Labrieville.[7]

Geology of the Site

The detailed geology of the Bersimis and Desroches dams is shown in plan on Fig. 3 and in section along the axes of the dams on Fig. 4. Beneath the Bersimis River and on the south bank of the Bersimis valley, bedrock outcrops or is overlain by thin patches of glacial till and lenses of sand, gravel, and boulders. However, beneath the north bank the bedrock surface dips downward to form what must have been a pre-glacial drainage channel. This depression in the rock has been infilled with a succession of till and varved clay and silt strata during glacial and interglacial periods. This complex of glacial materials reaches a maximum thickness of about 300 feet beneath the extreme north end of the dam and decreases thereafter as the rock surface rises to reach the surface some distance above the dam.

Of the several types of rock referred to previously, the only ones found within the immediate vicinity of the Bersimis and Desroches dams are the

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6. "Preliminary Report on Labrieville Area, Saguenay County," by M. Morin, P. R. No. 333, 1956, Dept. of Mines, Province of Quebec, 8 pp.
7. "Seismicity of the Earth and Associated Phenomena," by B. Gutenberg and C. F. Richter, Princeton University Press, 2nd ed., 1954, 310 pp.

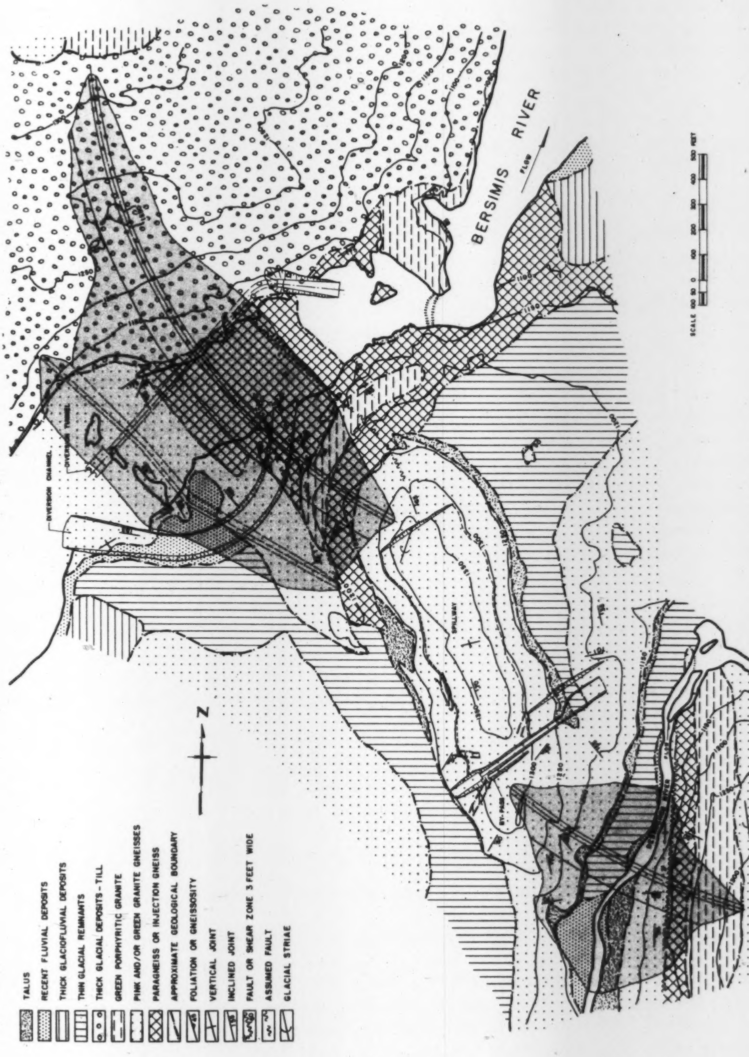
TALUS
RECENT FLUVIAL DEPOSITS
THICK GLACIOFLUVIAL DEPOSITS
THIN GLACIAL REMNANTS
THICK GLACIAL DEPOSITS - TILL
GREEN PORPHYRITIC GRANITE
PINK AND/OR GREEN GRANITE GNEISSES
PARAGNEISS OR INJECTION GNEISS
APPROXIMATE GEOLOGICAL BOUNDARY
FOLIATION OR GNEISSOSITY
VERTICAL JOINT
INCLINED JOINT
FAULT OR SHEAR ZONE 3 FEET WIDE
ASSUMED FAULT
GLACIAL STRIAE
N
BERSIMIS RIVER
FLOW
DIVERSION CHANNEL
DIVERSION TUNNEL
SPILLWAY
BY-PASS
1100
1150
1200
1250
SCALE 100 50 0 100 200 300 400 500 FEET

FIG. 3.—GEOLOGICAL PLAN

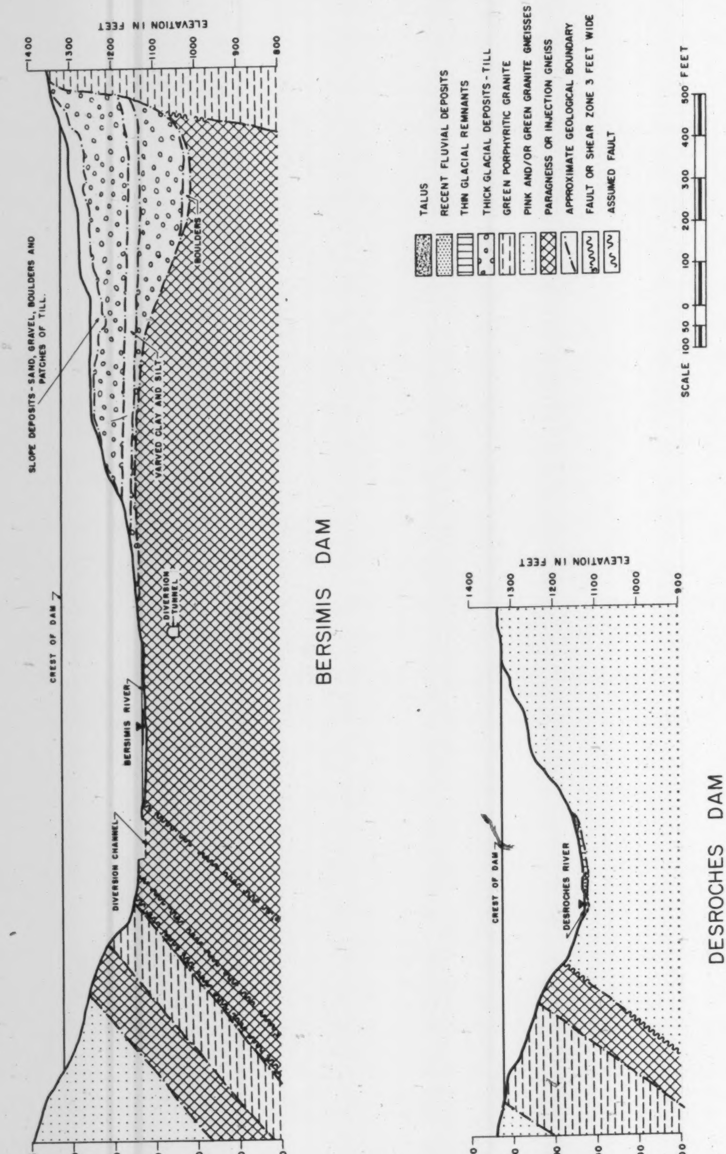
BERSIMIS DAM
ELEVATION IN FEET
1400
1300
1200
1100
1000
900
800
SLOPE DEPOSITS - SAND, GRAVEL, BOULDERS AND PATCHES OF TILL.
VARVED CLAY AND SILT
BOULDERS
CREST OF DAM
DIVERSION TUNNEL
BERSIMIS RIVER
DIVERSION CHANNEL
DESROCHES DAM
CREST OF DAM
DESROCHES RIVER
TALUS
RECENT FLUVIAL DEPOSITS
THIN GLACIAL REMNANTS
THICK GLACIAL DEPOSITS - TILL
GREEN PORPHYRITIC GRANITE
PINK AND/OR GREEN GRANITE GNEISSES
PARAGNEISS OR INJECTION GNEISS
APPROXIMATE GEOLOGICAL BOUNDARY
FAULT OR SHEAR ZONE 3 FEET WIDE
ASSUMED FAULT
SCALE 100 50 0 100 200 300 400 500 FEET

FIG. 4.—GEOLOGICAL SECTIONS ALONG AXES OF DAMS

paragneiss, pink and green granite gneisses, and green porphyritic granite.

The Bersimis dam is underlain for the most part by paragneiss although the upstream toe and cofferdam rest upon pink and green granite gneisses and the south abutment is cut by a zone of green porphyritic granite running parallel to the river. The foliation of the gneissic deposits strikes in the north-south direction and dips to the east at about 45 to 65 degrees. The major system of joints coincides with the orientation of the foliation while the minor jointing consists of two sets, one striking northeast-southwest and dipping about 55 to 75 degrees to the southeast, and the other striking northwest-southeast and dipping about 50 to 60 degrees to the southwest. The jointing is in general very tight and the amount of surface weathering of the rock was found to be extremely small.

A number of minor shear zones varying in width from 1 foot to 3 feet were found to occur along the dark bands of foliation. These shear zones were chloritized but contained no fault gouge and were very tight. One fault zone of more than minor significance was found running almost parallel to the downstream toe of the dam and dipping at about 85 degrees to the northeast, thus cutting across the foliation. The fault proper was about 3 feet wide and contained very dry, compact gouge. On both sides of it the rock was shattered for a width of about 5 feet, but the rock was, however, tight.

The extreme south end of the Bersimis dam is underlain by a mass of green granite gneiss which extends in a southeasterly direction to form the entire north abutment of the Desroches dam as well as the spillway area lying between the two dams. The southerly half of the Desroches dam is underlain by a succession of green and pink granite gneisses, paragneiss, and green porphyritic granite which occur as bands roughly paralleling the Desroches River. These rocks were exposed at the surface over much of the area of the Desroches dam, although a thin cover of glacially and fluvially deposited soils and talus existed near the river.

As in the case of the Bersimis dam the foliation of the gneisses strikes in the north-south direction and dips easterly at about 55 degrees. The major jointing coincides with the foliation and the minor jointing is similar to that which occurs in the Bersimis valley.

Structurally the rock in the Desroches valley proved to be in a very sound condition. Surface weathering was of extremely limited extent, and only one shear zone or fault of any significance was found. This shear zone of about three-foot width occurred along the contact of the granite gneisses and the paragneiss on the south bank, and it contained shattered and brecciated rock and fault gouge.

Availability of Materials

In general the Canadian Shield provides everywhere abundant supplies of excellent rock for use in rockfill dam construction. Surface weathering is usually very minor and quarries can be opened up with only small amounts of stripping being required. Almost all types of rocks occurring in the shield area are suitable for dam construction, although preference was given in the construction of the Bersimis dams to the granite gneisses rather than to the porphyritic granites and anorthosites as it was considered that the large crystal structure of the latter would lead to the production of a large amount of fines during quarrying. The locations of rock quarries are shown on Fig. 5.

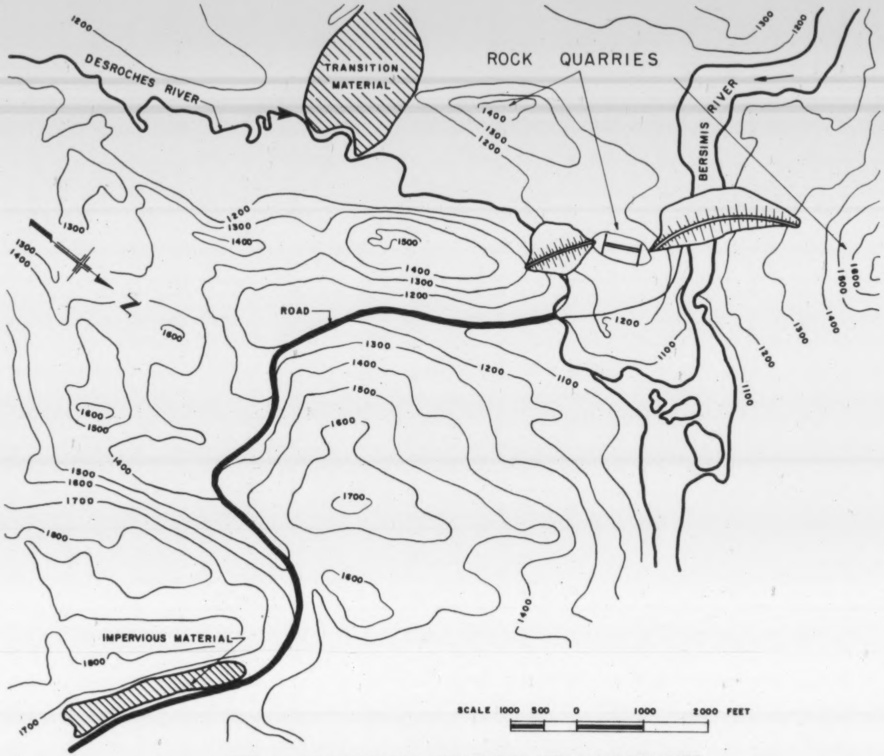
DESROCHES RIVER
TRANSITION MATERIAL
ROCK QUARRIES
BERSIMIS RIVER
ROAD
IMPERVIOUS MATERIAL
N
1100
1200
1300
1400
1500
1600
1700
1800
SCALE 1000 500 0 1000 2000 FEET

FIG. 5.—LOCATION OF QUARRIES AND BORROW PITS

Material for transitions is usually available within economic hauling distances from glacio-fluvial or recent fluvial deposits. Gravel deposits are usually fairly well graded and reasonably clean, but sand deposits, particularly when they occur as terrace formations, tend to be uniformly graded and on the fine side. For the Bersimis dams the transition materials were obtained from a gravel deposit located as shown on Fig. 5.

Materials suitable for use in impervious cores do not occur in abundance in this part of the Canadian Shield. Above the limit of marine clays at approximate elevation 400 feet, water-lain clays occur only as small patches of varved clays. Clayey glacial tills containing up to a maximum of about 10 per cent of clay particles occur in limited quantities and constitute the only impervious material in the region suitable for sloping-core construction. One such deposit was located as shown on Fig. 5, and it proved sufficient and satisfactory for the construction of both dams.

Exploration of Site

Aerial photography of the Bersimis area was already available at the beginning of the project and this was used at an early stage in the studies to obtain a general appraisal of the geology of the area. This preliminary work was followed by detailed geological mapping and subsurface explorations at the site. Subsequently additional photographic interpretation was done to locate deposits of suitable construction materials. While the borrow pits thus located were considered suitable for use, preference was given to those shown on Fig. 5, which were found by ground survey, since they were more economical to develop.

Subsurface explorations were begun at the dam sites in 1952 and carried on throughout 1953 and 1954. Diamond drilling was performed at both dam locations to determine the depth of overburden, the quality of rock, and the existence of major structural features. Diamond drilling was also undertaken in the river upstream from the Bersimis dam to determine the most suitable location for the diversion tunnel intake, and along the diversion tunnel line to prove the depth of rock cover above it. Standard AXT core was obtained in most of the drilling, although some EXT core was also obtained.

The presence of the deep overburden deposit on the north bank required additional explorations to determine the nature and the state of the soils forming it. These were done by standard diamond drilling methods and by churn drilling. The former method was found to be more satisfactory in penetrating the very dense glacial till and allowing bedrock to be reached at depths which sometimes exceeded 250 feet. Undisturbed soil sampling by the usual methods was found to be impossible because of the very dense and gravelly nature of the glacial till and the frequent occurrence of boulders within it. Disturbed tube samples were, however, recovered and from these the index properties of the foundation soils were determined. Six test pits were sunk to depths of about 50 feet to 60 feet to obtain chunk samples of the surface deposits for laboratory testing.

Geophysical methods were employed in the early stages of the explorations to determine the elevation of rock along two profiles through the north bank of the Bersimis River. The seismic method was used and, although it did produce profiles that were in general agreement with those subsequently determined by drilling, it was in error by as much as 45 feet in a 220-foot depth. For the most part the geophysically determined rock surface was higher than the actual one found by drilling.

Design

General Arrangement

The site chosen for the dams at Lac Cassé required the construction of a dam in the Desroches valley 1,100 feet long and 225 feet high, and a dam in the Bersimis valley 2,100 feet long and 200 feet high. The foundations for the Desroches dam were of sound rock and, therefore, this site provided ideal conditions for construction of any of several types of dam. The north bank of the Bersimis River, however, was covered with a thick mantle of overburden which precluded the consideration of a mass concrete dam for that site. It was believed that an earth or rockfill dam would be most suitable for construction on the overburden and, after investigation of the availability of materials, a decision was made in favour of the rockfill type. Economic studies also indicated that this type of dam would be the best choice for construction in the Desroches valley. Therefore, the rockfill type was adopted for both dams.

The decision in favour of the sloping-core type of rockfill dam was made after consideration of the construction methods which would be most adaptable at the site. It was believed that the sloping-core design offered the following advantages:

(1) It allows for the placing of rock in high lifts. As a result a minimum of building and rooting up of roads is required, and rock limited in size only by the loading and transporting equipment may be used.

(2) It allows for the placing of a large volume of the main body of the dam as a separate and independent operation. This is a definite advantage where core construction may be delayed because of adverse weather conditions.

The general arrangement of the dams may be seen on Fig. 6. The common rock abutment was excavated to form an overflow spillway, and a gate section was subsequently constructed at one end of the spillway.

The dams were arched upstream so that the hydrostatic reservoir load would tend to induce compressive forces in the cores.

Typical sections of the two dams are presented on Figs. 7 and 8. These indicate the design of sections based on both rock and earth foundations.

Foundation Materials

With the exception of the north bank of the Bersimis River, the foundations of both dams were stripped down to bedrock, the various types of which are shown on the geological plan on Fig. 3. In the north abutment of the Bersimis dam the design required the impervious core to be carried down in a trench to a relatively impervious stratum. Over much of its length this trench was cut into the surface of the upper till layer shown on Fig. 4. This till layer is approximately 100 feet in thickness and is underlain by a fairly continuous layer of varved clay and silt of about 25 feet in thickness and a lower till layer which is about 100 feet thick and located beneath the extreme north end of the dam.

Both till layers have similar properties and for design purposes no distinction was made between them. The tills together with the interposed varved clay layer exist naturally in a very dense state, and it is believed that they have been overconsolidated to a moderate degree. The ranges of particle size

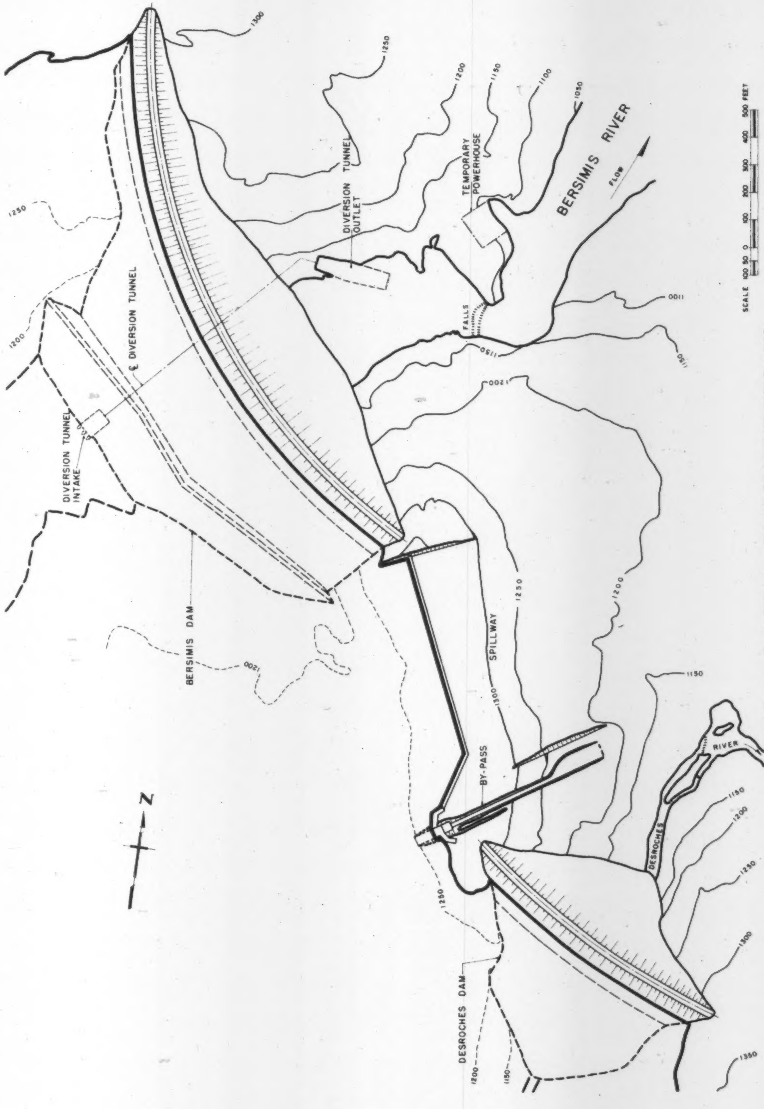
1300
1250
1200
1150
1100
1050
DIVERSION TUNNEL OUTLET
TEMPORARY POWERHOUSE
BERSIMIS RIVER
FLOW
FALLS
℄ DIVERSION TUNNEL
DIVERSION TUNNEL INTAKE
BERSIMIS DAM
SPILLWAY
BY-PASS
DESROCHES RIVER
DESROCHES DAM
N
1350
SCALE 100 50 0 100 200 300 400 500 FEET

FIG. 6.—GENERAL ARRANGEMENT OF DAMS

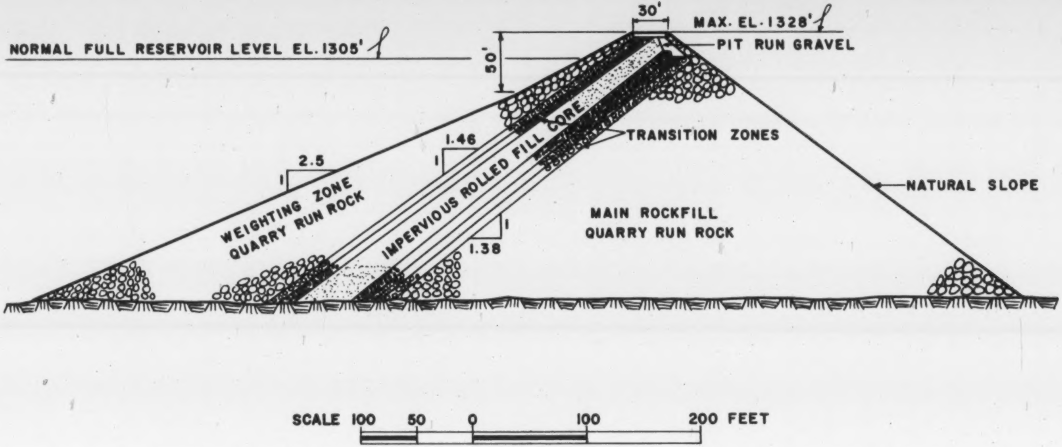
NORMAL FULL RESERVOIR LEVEL EL. 1305'
30'
MAX. EL. 1328'
PIT RUN GRAVEL
50'
TRANSITION ZONES
1.46
1
2.5
1
WEIGHTING ZONE
QUARRY RUN ROCK
IMPERVIOUS ROLLED FILL CORE
1
1.38
MAIN ROCKFILL
QUARRY RUN ROCK
NATURAL SLOPE
SCALE 100 50 0 100 200 FEET

FIG. 7.—BERSIMIS AND DESROCHES DAMS, TYPICAL SECTION

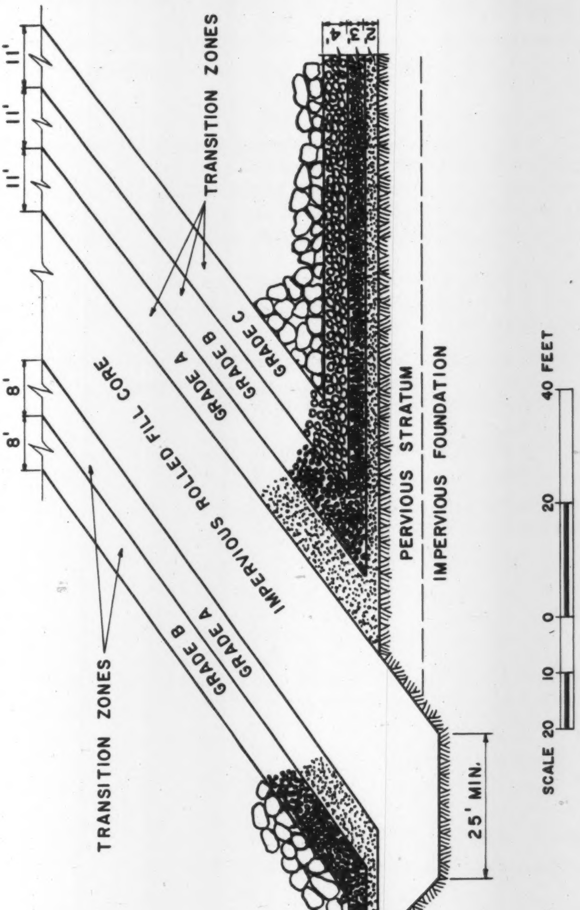
TRANSITION ZONES
8'
8'
11'
11'
11'
TRANSITION ZONES
GRADE B
GRADE A
IMPERVIOUS ROLLED FILL CORE
GRADE A
GRADE B
GRADE C
2'
3'
4'
PERVIOUS STRATUM
PERVIOUS STRATUM
IMPERVIOUS FOUNDATION
25' MIN.
SCALE 20
10
0
20
40 FEET

FIG. 8.—BERSIMIS DAM, PARTIAL SECTION ON OVERBURDEN

gradations for the two materials are shown on Fig. 9 and the other know soil properties for them are summarized in Table 1. The values of the effective shear strength parameters c' and ϕ' were assumed for the purposes of calculation and are considered to be reasonable and slightly conservative.

Dam Materials

Rock for the main rockfills and the upstream weighting zones of both dams consisted almost entirely of sound green granite gneiss. The maximum rock size allowed was governed by the capacity of the moving equipment, while the minimum size was rock dust which was not allowed to constitute more than 15 per cent of any load. Soil and unsound rock were not allowed to be placed in the rockfill.

The impervious core is protected by three transition zones on the downstream side and two on the upstream side. The material gradations that were specified were as follows:

Grade A - Ranging in size from not more than 2 per cent passing a No. 200 sieve to that passing a 1/4-inch sieve.
Grade B - Gravel ranging from 1/4-inch minimum to 3 inches maximum size.
Grade C - Rock ranging from 3 inches minimum to 10 inches maximum size.

Clean gravel was screened and the transition materials thus produced had gradation ranges as shown on Fig. 10.

TABLE 1.—PROPERTIES FOR NORTH BANK SOILS OF BERSIMIS DAM

Property	Dense Till	Varved Clay and Silt
Specific gravity	2.73	2.73
Natural water content, in %	10 - 20	18 - 25
Natural bulk density, lb/cu ft	127 - 145	125 - 134
Coefficient of permeability - cm/sec	10^{-2} to 10^{-5} Avg = 10^{-4}	10^{-6}
Effective shear strength parameters:		
c' - psf	720	720
ϕ' - degrees	35	35

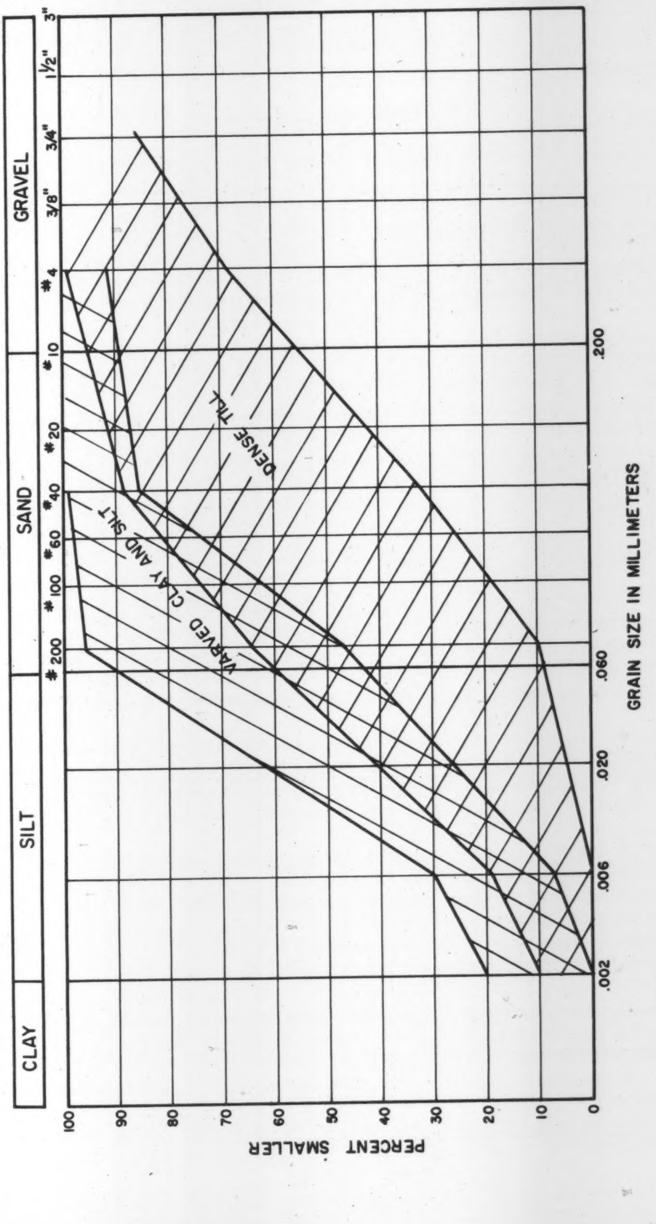

FIG. 9.—BERSIMIS DAM NORTH ABUTMENT, PARTICAL SIZE DISTRIBUTION OF FOUNDATION MATERIALS

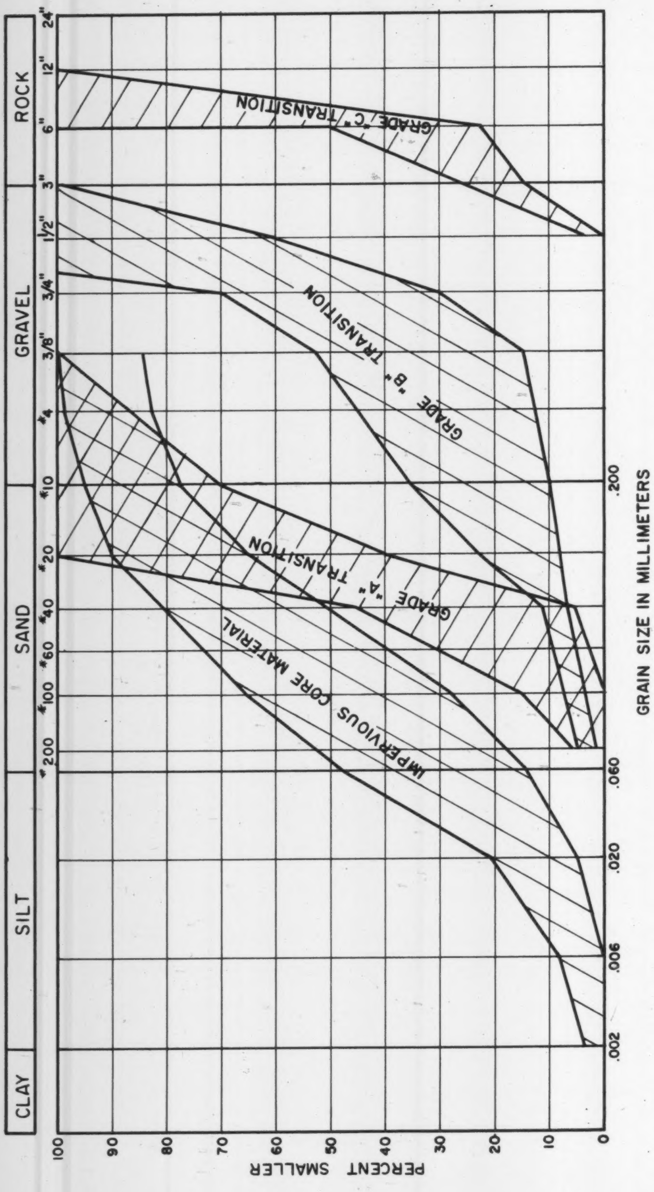

FIG. 10.—PARTICLE SIZE DISTRIBUTION OF IMPERVIOUS CORE AND TRANSITION MATERIALS

The impervious core material was a well-graded glacial till with the following fraction proportions:

Clay	0 to 5 per cent
Silt	10 to 45 " "
Sand	30 to 80 " "
Gravel	5 to 25 " "

The range of gradings is shown on Fig. 10 and its relevant properties are listed in Table 2.

The coefficient of permeability was found by laboratory testing to be approximately 10^{-6} cm/sec and this was considered sufficiently low for use in core type dams such as the Bersimis and the Desroches. This permeability was also confirmed by a large scale field test in which water under a head of 450 feet was forced through a sample of the till, 18 inches in diameter and 30 inches thick, for a period of three weeks.

Triaxial shear testing was performed on 4-inch diameter samples of the till material smaller than the No. 4 sieve compacted at the mean placement water content and to the mean placement dry density. These tests were performed under undrained conditions and pore pressure measurements were made during all tests. The results of these tests indicated that the build-up of pore pressures during construction would not exceed 20 per cent of the superimposed load at any point. Laboratory tests also indicated that the dissipation of pore pressures would occur very slowly, the value of c_v obtained in these tests being 3.2×10^{-3} cm^2/sec.

Stability Analyses

Stability analyses were made on the dams to ensure that no failure by sliding would occur during any stage of construction or period of future operation. The analyses were made in terms of effective stresses and were two-dimensional in scope. The geometrical shape and the location of the potential failure surface were allowed to vary and the slices method of analysis was employed.

The relevant properties of the foundation and dam materials are contained in Tables 1 and 2. In addition to these it was assumed that the rockfill and the transition zone materials were free draining and that the angle of shearing resistance (ϕ) for these materials was 42 degrees at a relative density of 70 per cent.

The types of failure considered are shown on Fig. 11. Type I failure was considered at the point of maximum height of dam, while Type II was only possible where the Bersimis dam rested upon the north bank deposits. Type III is a downstream failure similar to Type II which could only occur on the Bersimis dam north bank. For a downstream failure similar to Type I, the minimum factor of safety is 1.0 for which the failure surface is the downstream slope of the dam.

Three conditions of loading were considered, the construction case, the steady seepage case, and the drawdown case, and the minimum factors of safety found are contained in Table 3. For the construction case, the pore pressure parameter B was assumed to be 0.4 for the foundation materials and was taken from the triaxial test data to be 0.2 for the impervious core material.[8,9] Dissipation of pore pressures during construction was considered to be negligible. The analysis was performed

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[8] "The Pore-Pressure Coefficients A and B," by A. W. Skempton, Géotechnique, Vol. IV, 1954, pp. 143-147.

[9] "The Use of Pore-Pressure Coefficients in Practice," by A. W. Bishop, Géotechnique, Vol. IV, 1954, pp. 148-152.

TABLE 2.—PROPERTIES OF IMPERVIOUS CORE MATERIAL FOR BERSIMIS DAMS

Specific gravity	2.73
Optimum water content, in % (Fraction $<$ No. 4 sieve)	9.3
Mean placement water content, in % - (Fraction $<$ No. 4 sieve),	9.6 = 1.03 x Optimum
Maximum dry density by standard Proctor compaction test (Corrected for material $>$ No. 4 sieve), lb/cu ft	135
Mean placement dry density lb/cu ft	132 = 0.98 x Proctor maximum
Coefficient of permeability, cm/sec.	1 x 10^{-6} to 4 x 10^{-6}
Effective shear strength parameters:	
c', psf, construction case	1,300
c', psf, steady seepage and drawdown cases	500
ϕ', degrees	36

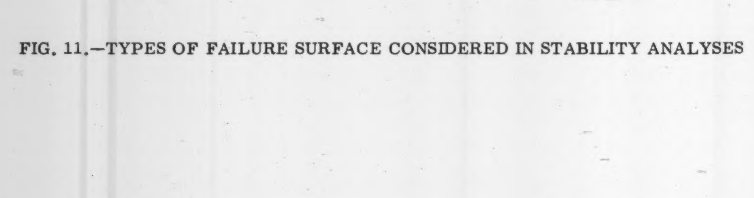

TRANSITION ZONES
IMPERVIOUS CORE
WEIGHTING ZONE
MAIN ROCKFILL
TYPE I
TYPE II
TYPE III

FIG. 11.—TYPES OF FAILURE SURFACE CONSIDERED IN STABILITY ANALYSES

for the condition when the dam was completed and the reservoir empty, and as it was stable for this condition a failure at an intermediate stage in construction was impossible. The analysis was then repeated for varying reservoir levels, and the minimum factor of safety of 1.80 was found to occur with a Type II failure and the reservoir elevation below 1170 feet. The variation of factor of safety with reservoir level is shown on Fig. 12. In reality the reservoir elevation rose during the construction of the dam and the effect of this was to produce actual factors of safety greater than those shown in Table 3.

In the analysis for the steady seepage case, it was considered that the minimum probable operating reservoir level would be 1275 feet and that steady seepage conditions would not be set up with reservoir levels lower than this. Complete saturation of the impervious core was assumed to have occurred when the steady seepage flow net became established. The pore pressures were determined from the latter and a decrease in c' due to saturation of the fill was allowed for. For these conditions the minimum factor of safety of 2.15 was found for a Type I failure with the reservoir elevation at 1275 feet.

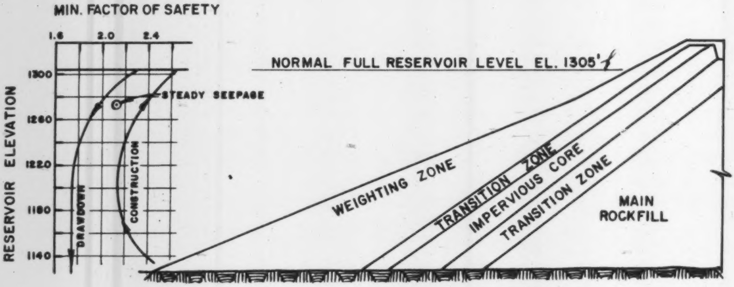
For the drawdown case it was assumed that steady seepage conditions had been established before drawdown occurred. Therefore, $\bar{B}$ was assumed to be 1.0. Drawdown to various levels was considered to take place instantaneously and the minimum factor of safety of 1.70 for a Type II failure was found to occur when the reservoir is lowered below elevation 1175. The variation of factor of safety with drawdown elevation is shown on Fig. 12.

Settlement

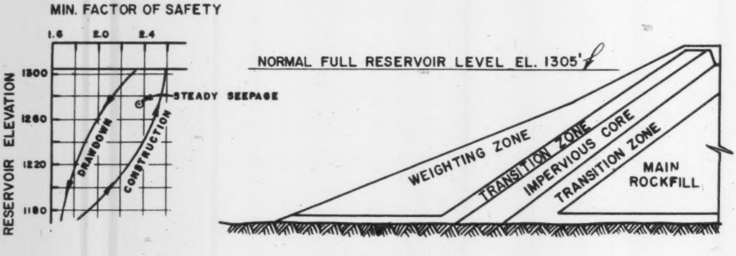
In the design of the Bersimis dams it was realized that settlement would occur and that an allowance should be made for it. However, because of the

TABLE 3.–MINIMUM FACTORS OF SAFETY FOR STABILITY AGAINST SLIDING

Stability Case	Failure Type		
	I	II	III
Construction .... (Dam completed and reservoir rising)	2.15	1.80	2.55
Steady seepage .... (Failure Reservoir) (Type Elevation) ( ) (I & II 1275 feet) (III 1312 feet)	2.15	2.35	2.25
Drawdown........................ (Minimum values for drawdown to elevation 1175 or lower)	1.75	1.70	Condition Unaffected by Drawdown


MIN. FACTOR OF SAFETY
1.6
2.0
2.4
RESERVOIR ELEVATION
1300
1260
1220
1180
1140
STEADY SEEPAGE
DRAWDOWN
CONSTRUCTION
NORMAL FULL RESERVOIR LEVEL EL. 1305'
WEIGHTING ZONE
TRANSITION ZONE
IMPERVIOUS CORE
TRANSITION ZONE
MAIN ROCKFILL

FAILURE SURFACE TYPE I


MIN. FACTOR OF SAFETY
1.6
2.0
2.4
RESERVOIR ELEVATION
1300
1260
1220
1180
STEADY SEEPAGE
DRAWDOWN
CONSTRUCTION
NORMAL FULL RESERVOIR LEVEL EL. 1305'
WEIGHTING ZONE
TRANSITION ZONE
IMPERVIOUS CORE
TRANSITION ZONE
MAIN ROCKFILL

FAILURE SURFACE TYPE II

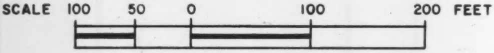

SCALE 100 50 0 100 200 FEET

FIG. 12.—VARIATION OF FACTOR OF SAFETY WITH RESERVOIR ELEVATION

sloping-core nature of the dams it was evident that the settlement of the dam crests would result from movements in the main rockfill, and that such settlements cannot be rationally calculated but must be estimated from previous experience. It was assumed that settlement in the rockfill would amount to about 5 per cent of its height and that at least two-thirds of this would occur during construction. To accommodate the movements expected after construction, ample superelevation was included in the design.

Foundation settlement is confined to the north bank deposit of the Bersimis dam, but because of the nature of these soils good samples for consolidation testing were very difficult to obtain and this precluded any accurate settlement calculation. However, a rough estimate, based on limited consolidation test data, showed that the foundation settlement would be very small. Since only a small portion of this would occur after construction, no additional allowance in superelevation was made.

Seepage and Transition Design

Seepage from the reservoir in the immediate vicinity of the Bersimis dams was considered to be possible directly through the core, through the bedrock beneath or around the dams, and through the north bank deposits. Seepage through the core was estimated, on the basis of the coefficient of permeability shown in Table 2, to be of the order of 5×10^{-2} cfs for the Bersimis and 2.5×10^{-2} cfs for the Desroches dam. Seepage through the surrounding rock mass was considered to be of much greater magnitude than seepage through the core, and to reduce this flow of water extensive cement grouting was carried out beneath the impervious core where it rests upon rock. Blanket grouting to a depth of 15 feet was done on a 10-foot square grid spacing over this entire area, and in addition to this a grout curtain was constructed extending into rock from 50 to 60 per cent of the acting head of water. As the surrounding rock abutments were composed of sound rock, no additional abutment grouting beyond the limits of the dams was deemed necessary.

During the design of the Bersimis dam, the possibility that the north bank foundation might prove to be more pervious than was considered desirable became a matter of concern. This concern was increased during excavation of the north bank core trench when some evidence of stratification amongst the more pervious parts of the deposit was uncovered. However, the impervious core was carried down to depths as great as 50 feet, and it was considered that any seepage passing beneath the core would be of a minor nature. As an added precaution, an attempt was made to seal any underlying pervious layers or lenses by bentonite injections. Grout acceptance by the foundation materials was very small.

Transition zones on both the upstream and downstream sides of the impervious core were designed and specified as outlined previously. These gradation specifications satisfy the filter requirements as outlined by Karpoff,[10] and the materials actually produced satisfied these requirements with the exception of the finer portion of Grade B material. To check on the possibility of particle migration with the actual materials a large size permeability test was conducted at the site with an effective hydraulic gradient of 180. After three weeks operation the steel pipe permeameter was cut open and the transition materials were examined, but no evidence of migration was found. On the basis of this test and in view of the fact that the maximum hydraulic gradient in either dam is less than 10, it was concluded that the transitions as constructed would perform satisfactorily. The thicknesses of the transition zones were determined on the basis of the minimum widths which were considered practicable.

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[10] "The Use of Laboratory Tests to Develop Design Criteria for Protective Filters," by K. P. Karpoff, Proceedings, ASTM, Vol. 55, 1955, pp. 1183-1198.

Construction

Schedule and Progress

The schedule required construction to be started in the fall of 1953 and to be completed before the end of 1955. In keeping with this schedule, the contractor set up a camp near the dam sites and began stripping the foundations and quarry areas in late 1953. Grouting of foundations was also started before the end of the year. The cofferdams, diversion tunnel and diversion intake were completed by the spring of 1954.

The rockfill operation was begun in the summer of 1954 and proceeded according to schedule, but the progress on impervious core construction was slow due to unusually wet weather. The 1955 season was much more favourable for the construction of earth fill, and the dams were completed, as scheduled, before the end of the year.

Diversion

The diversion of the Bersimis River flows was complicated by the necessity to supply water to a temporary hydroelectric plant installed at the head of the falls immediately downstream from the dam. A diversion tunnel 39 feet wide by 36 feet high was constructed beneath the river bed, and a rockfilled timber crib weir was constructed at the outlet to maintain a forebay for the temporary power house. A channel was also excavated to divert the river flows until the diversion tunnel was completed. The upstream and downstream cofferdams were constructed of rockfill with impervious blankets.

The diversion intake was provided with two gates 19 feet wide by 34 feet high and capable of operating under a head of 180 feet. These were closed at the time scheduled for filling the reservoir. The intake also contained a 102-inch diameter discharge regulator, which was operated to discharge sufficient water through the diversion tunnel during the reservoir filling period to operate the temporary power plant. After the Bersimis power house was placed in operation in late 1956, the discharge regulator was closed and a concrete plug was placed in the tunnel beneath the impervious core of the dam.

Diversion of the Desroches River flows was effected by constructing a concrete culvert along the bottom of the steep right bank.

Foundation Treatment

With the exception of the north abutment of the Bersimis dam, the design of both dams required that the foundations be stripped to rock. In the case of the Bersimis dam, however, the main body of rockfill was dumped on a boulder field overlying the bedrock in the river bed, and the upstream cofferdam which formed the base of the loading zone was constructed at its north end on relatively impervious overburden in the river bed.

The north bank of the Bersimis River was stripped of topsoil, and the earth abutment was cut to a regular slope to receive the base transition. The cutoff in the north bank was carried deeper than original plans called for due to the occurrence of pervious lenses in the foundation.

Small volumes of rock were excavated in the core foundation area, especially at the Desroches Dam, to correct adverse slopes. Concrete was placed in some areas where uneven rock would make it difficult to compact impervious fill.

Special treatment at the Desroches dam was necessary in a shear zone intersecting the foundation of the impervious core. The gouge was removed to a depth of about 3 feet from the upstream to the downstream limits of the core, and the trench formed was filled with concrete. A shaft was also sunk along the dip of the shear zone to a depth of about 20 feet, and this was filled with concrete to form a cutoff.

Guniting of the rock surface prepared for the impervious core was carried out over a considerable area. The gunite was effective in sealing open joints during the blanket grouting operation.

Quarrying

The two main quarries were located in a hill just upstream of the two dams. During the 1954 construction season a supplementary quarry was operated on the north bank of the Bersimis, using the coyote hole method. In 1955 a quarry was also opened on the spillway site between the two dams. With the exception of the coyote hole operation on the north bank of the Bersimis, all quarrying was carried out by the benching method.

The following is a list of equipment used for quarrying and transporting rock:

Drills

2 Quarrymasters
5 Nine-inch churn drills

Shovels

1 - 5 yard
2 - 3-1/2 yard
4 - 2-1/2 yard

Trucks

14 - 35 ton
12 - 22 ton

Borrow Pits

A screening plant was set up about one-half mile from the source of transition materials, and the gravels were separated into the three grades required for the dams. Since the pit lay in the area to be flooded, the transition materials were screened during the early stages of construction and stockpiled at the screening plant about two miles from the dams.

The impervious material borrow pit was located on a hillside beside the main road about four miles from the dams. The material, a glacial till, was very dense in the natural state and there was no problem of excess moisture in the undisturbed materials in the pit even after heavy precipitation. The material was excavated in benches using power shovels.

Rockfill

Rockfill operations for the main body of the two dams were carried out by end dumping from the fills as they were advanced from the abutments. The fills were built from the north bank of the Desroches dam and, with the exception of the rock obtained from the coyote operation, from the south bank of the Bersimis dam.

Rock was placed in both dams in high lifts and a maximum height of 140 feet was reached in the Desroches dam. Segregation of the rock was apparent where the lift was high and, with the large rock concentrated at the bottom, special attention was required in the placing of the transitions, as mentioned later in this paper. The lifts were advanced with a minimum top width of 60 feet. This dimension was considered a minimum for efficient operation.

During the 1954 construction season the main body of rockfill on both dams was carried up to elevation 1240 with a top width of 60 feet. In the first stage of construction in the 1955 season the main rockfill was extended to its full width at elevation 1240. In the next stage the fills were raised to elevation 1280 where the final width of the main body is approximately 60 feet. In the final stages the rock was raised from elevation 1280 to the crest.

The surface preparation prior to placing succeeding lifts in the main rockfill body was done by bulldozers and rooters, and on some occasions by back hoes. Thorough sluicing of the surface was carried out after rooting. Considerable work was involved in preparing the top of a lift for the placing of a succeeding lift, and this confirmed one of the design considerations in the choice of the sloping-core type dam.

In contrast to the method of constructing the main body of rockfill, the upstream weighting zone was brought up in shallower lifts usually following the constructing of the impervious blanket and transition materials. Very little surface preparation was done on the weighting zone lifts.

Sluicing of the rock placed in the main body was carried out from cantilevered platforms located at each corner of the dumping face. Under a pressure of 125 psi the jets from the monitors were trained back toward the dumping face and met the rock as it first came in contact with the slope. This operation is shown on Fig. 13. The volume of water used was generally equivalent to four times the volume of rock placed.

Special precautions were exercised in dumping rock on the north bank of the Bersimis dam. Since the overburden was covered with transition materials, it was necessary to avoid the displacement of this material by falling rock. The rock was dumped against the abutment rather than down the bank at this location.

An attempt was made to place a well graded fill throughout by classifying the rock size in the trucks as they left the quarry. With the high lifts used in the lower parts of the dams however, it was difficult to prevent segregation of the rock and the practice met with limited success. On the basis of experience gained with the high lifts, it is believed that there is some advantage in specifying the placing of a finer rock in an upstream zone in order to limit the size of voids adjacent to the transition layers.

Transition

Transition materials were used both upstream and downstream of the core and also under the rockfill on the north bank of the Bersimis dam. These materials were spread and compacted by bulldozer. To prevent bulking of the

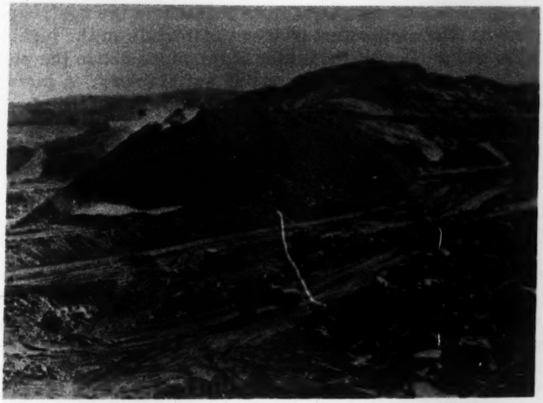

FIG. 13.—BERSIMIS DAM, ROCKFILL

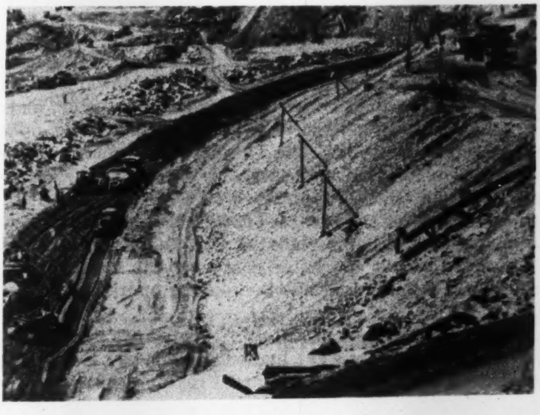

FIG. 14.—PIPE FOR PLACING TRANSITION MATERIAL

fine transition material, excess water was added during the compacting operation.

Due to traffic problems across the impervious fill, an alternative method of placing the coarsest transition was adopted for the zone on the downstream side of the impervious core. The alternative method consisted of dumping the materials through 26-inch diameter pipes located on the upstream slope of the main rockfill. The momentum of these materials in the pipes was checked by heavy chains which hung at the outlet of the pipes above the placing level. This alternative was found to be reasonably satisfactory. A picture of the arrangement of a pipe on the rockfill slope is shown on Fig. 14.

As stated previously, segregation in high lifts in the rockfill caused concentration of large rock at the bottom of the slopes. This necessitated special placing of the coarsest transition material by hand in filling some of the larger voids.

Impervious Core

In general, the impervious core material was transported by truck from the borrow pit, dumped on the fill, and spread by bulldozers. During the period of greatest congestion in 1955, however, trucks were driven on to a ramp on the Bersimis dam and emptied into scrapers. These were then driven on to the fill and the material was dumped and spread in the same operation. In this way trucks hauling the earth fill did not interfere with the placing and compaction of impervious fill.

The impervious fill was spread in layers 8 inches thick. Boulders exceeding 5 inches in diameter were removed from the fill by hand after the spreading operation, and these were dumped in the coarse transition zone or in the upstream weighting zone. A large number of boulders were removed from the fill in this way.

Addition of a small amount of moisture was frequently required on the impervious fills, and it was added by spraying with hoses during the spreading operation. The water content was maintained below the standard Proctor optimum during the 1954 construction season when the lower part of the dam was constructed. During the final construction season however, as the fill rose to higher elevation, the water content was kept at or above the optimum.

Since construction of the impervious zone fell behind schedule in 1954, placing was continued as late as possible in the autumn season. As early as mid-October freezing weather interfered with operations during the night shift and salt was utilized on several occasions to make it possible to continue the placing operation. Calcium chloride was dissolved in the water which was sprayed on the fill to bring the water content to the specified value, and this usually made it possible to continue the placing operation with up to 12 degrees of frost.

Compaction of the impervious fill was carried out by means of pneumatic-tired vehicles, which usually were loaded tandem-wheeled, 10-ton hauling units. Compaction very near to standard Proctor density was maintained throughout.

Due to the grading of the impervious core material, it was not practicable to attempt to carry out density tests in accordance with some of the established procedures. Instead a method was adopted which involved the excavation and transportation to the field laboratory of a chunk of compacted fill. A specimen was broken from this mass, weighed, and then immersed in oil to determine its volume.

Fig. 15 shows an over-all picture of the Bersimis dam under construction.

Summary of Quantities and Rates of Construction

Table 4 contains a summary of the quantities of materials used in construction of the dams.

TABLE 4.—SUMMARY OF QUANTITIES IN CUBIC YARDS

Material	Bersimis Dam	Desroches Dam	Total
Rockfill	2,575,000	1,081,000	3,656,000
Transition......	713,000	222,000	935,000
Impervious	383,000	124,000	507,000
	3,671,000	1,427,000	5,098,000

The weekly rates of placing materials were as follows:

Rockfill	80,000 cu yd
Transition	30,000 cu yd
Impervious	15 - 20,000 cu yd

Performance

Reservoir Elevations

Filling of the Lac Cassé reservoir began in 1955 and by the end of construction in November, 1955 its elevation was 1170 feet. Continued storage of water steadily raised the elevation to its normal maximum level of 1305 feet by November, 1957. The level was then gradually drawn down to elevation 1288 feet by late April, 1958, and since the commencement of the spring flood it has now risen again to 1305 feet.

Settlement Records

Settlement monuments were established on the crests of both dams in early 1956 for the measurement of vertical and horizontal movements. Between May, 1956 and February, 1958 maximum settlements of 1.9 inches and 1.8 inches respectively have been recorded in the Bersimis and Desroches dams. In both dams the settlements have varied directly with the height of fill. Horizontal movements during this period have been negligible or within the limits of instrumental error.

Seepage

During and subsequent to the filling of the reservoir a close observation has been maintained of the seepage emerging at the downstream toes and

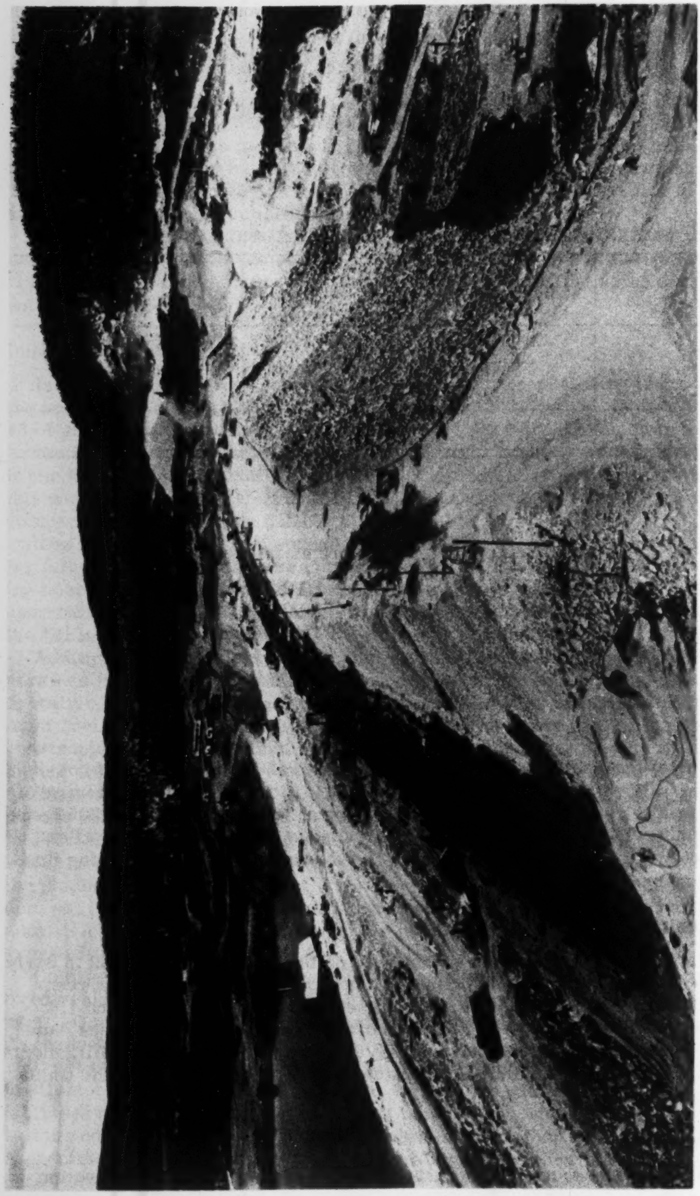

FIG. 15.—BERSIMIS DAM - GENERAL VIEW SHOWING CONSTRUCTION

abutments. A very small amount of seepage has occurred through the south rock abutment of the Bersimis dam but the quantity is not considered to be injurious and no attempt has been made to stop it. The seepage through the Bersimis north bank has not materialized to a discernible degree, and on the basis of the experience gained thus far it is believed that no seepage problem will develop there. The existence of seepage through the dam proper is difficult to establish, but no indication of abnormal water percolation through either dam has yet appeared.

ACKNOWLEDGMENT

The authors are grateful to the Quebec Hydro-Electric Commission for permission to present this paper and to its Engineers who have supplied much of the information concerning the Bersimis dams. The authors also wish to thank H. G. Acres & Company Limited and their colleagues for their assistance in the preparation of this paper.

DISCUSSION

F. L. LAWTON,[11] **F. ASCE.**—The paper is a valuable contribution to the literature on rockfill dams. The authors have treated the geological aspects of the sites fully, as warranted by the deep overburden on the north abutment of the Bersimis dam.

This deep overburden constitutes, perhaps, the most outstanding feature of the Bersimis dam. The relative grain size of the material contained in the varved clay and silt strata as contrasted with the glacial till is noteworthy. It would be interesting to know if the field investigations turned up any evidence of instability of the material from the strata of varved clay and silt, in view of the behaviour of much material of a somewhat similar nature found throughout the Canadian Shield, which can best be described as being a rock flour. R. F. Legget[12] has drawn attention to the properties of these materials and indicated their highly impervious nature if consolidated, confirmed also by the authors' finding for the co-efficient of permeability at 10^{-6} cm/sec. Legget has also noted their extreme instability in the presence of water in excess of the liquid limit. In this connection it is worth noting the authors' reference to the "overconsolidated" nature of the overburden on the abutment.

The authors' experience with the seismic method employed for the geophysical survey of the north bank of the Bersimis, with its maximum error of 45 ft. in a 220 ft. depth is to be noted; the geophysical survey indicating the rock surface to be higher than that found by drilling. Similar results have been found with other methods of geophysical surveys and is not peculiar to Canadian conditions, analogous results having been secured in tropical countries. The geophysical methods are useful but must be proven by actual diamond drilling.

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[11] Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

[12] "Soil Engineering at Steep Rock Iron Mines, Ontario, Canada," by Robert Ferguson Legget, Proceedings, Institution of Civ. Engrgs., Paper No. 6304, Vol. II, October, 1958, pp. 169-188.

The authors refer to the special precautions exercised in dumping rockfill on the north abutment of the Bersimis dam, relating the need for this to the transition materials covering the overburden and the necessity for avoiding displacement of this material by falling rock. The authors would add to the value of the paper if they indicated how the sluicing water was handled.

The usage of calcium chloride for the construction of the impervious zone as late as possible during the autumn season is an interesting confirmation of a procedure which has been adopted elsewhere. Can the authors state if they have found anything detrimental in such a practice?

F. W. PATTERSON,[13] and D. H. MacDONALD,[14] M. ASCE.—The authors wish to thank Mr. Lawton for his discussion and in reply, to offer the following comments:

The layer of varved clay and silt which was found between two thick layers of glacial till in the north bank deposits is appreciably coarser in grain size than the varved clays found at Steep Rock Lake and described by R. F. Legget.[12] Whereas the percentages of clay size particles in the Steep Rock deposit range from about 25 to 85, the similar values for the Bersimis deposit range from 0 to 20 per cent. Moreover, the entire north bank deposit of till and varved clay existed in an overconsolidated state with a low sensitivity, in contrast to the normally consolidated state and the highly sensitive structure of the Steep Rock clays. Because of this absence of high sensitivity, none of the unstable conditions which were found at Steep Rock Lake, were observed or would be expected to occur upon remoulding of the Bersimis varved clay.

In the construction of the main rockfill on the north bank care was exercised to ensure that the transition layers separating the rockfill from the foundation deposits were not damaged. The rockfill immediately overlying these transitions was built with the front of the stage progressing towards the north bank. Where this face met the steep portion of the abutment bordering the river it was necessary to support the transition layers on the steep slope with a berm of rockfill placed and compacted in thin lifts and sluiced only in part. This berm provided stability for the transition layers and provided access for their construction. Aside from this small portion the entire rockfill was sluiced in the standard manner. The sluicing water percolated down through the rockfill and on the north bank it flowed directly into the river or it was collected in a ditch along the downstream toe which was lined with layers of transition material. Although the water flowed over and through the transition layers the sluicing jets were not played directly on them. This operation did not result in any known detrimental effects.

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[13] Sr. Engr., Hydr. Div., H. G. Acres & Co. Ltd., Niagara Falls, Canada.

[14] Geotechnical Engr., Technical Div., H. G. Acres & Co. Ltd., Niagara Falls, Canada.

The use of calcium chloride in the construction of the impervious core did not, to the best of the authors' knowledge, adversely affect the quality of the core in any way. The densities obtained during compaction were as high when calcium chloride was used as they were without it. In view of the fact that the till used for the Bersimis core contained no more than about 5 per cent of clay size particles it is reasonable to conclude that the calcium chloride would not change the structure of the compacted soil. However, in the case of impervious materials containing much higher quantities of clay size particles, and particularly in cases where these particles may be highly active clay minerals, the use of calcium chloride followed by a leaching process during a steady seepage state might result in the creation of a sensitive soil structure within the core. Although it is possible that even in such cases the use of calcium chloride may not prove to be detrimental, the authors would not like to recommend this practice in these cases without studying the possible effects by laboratory testing.

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Paper No. 3082

ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM

By Torald Mundal,[1] F. ASCE

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SYNOPSIS

This paper describes the design and construction of the rockfill dam portion of Brownlee Hydroelectric Project. Of unusual interest are: diversion of flood waters over the partially completed embankment; inclusion of large quantities of compacted small rock material in the dam; and construction of the embankment on 110 feet of river-deposited materials.

INTRODUCTION

Three hydroelectric projects are currently scheduled by Idaho Power Company to ultimately develop 1,185,000 kilowatts of power by utilizing a 602-foot drop in the Hells Canyon reach of the Snake River along the Oregon-Idaho border. These projects, in downstream order, are Brownlee, Oxbow, and Hells Canyon. (Fig. 1) Brownlee and Oxbow projects, presently under construction, have completion dates of 1958 and 1960, respectively.

Brownlee Hydroelectric Project is situated in the upstream end of the 6,000-foot deep Snake River gorge, very appropriately named "Hells Canyon." The rugged terrain presented challenging design and construction problems, solutions of which resulted in several innovations in the field of rockfill dams. Project features other than the dam are discussed only because of some direct relationship with the dam.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1734. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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Basin Description

The 72,000-square mile drainage basin upstream from the damsite is divided into three distinct areas. The upper area rises to a maximum elevation of 13,700 feet on the peak of Grand Teton. Precipitation falling on this area builds an annual snow pack which supplies a major portion of the summer flow of the Snake River.

The middle area contains many acres of irrigated farms as well as forests and brushland. This area, with its vast underground storage reservoirs, collects much of the annual rainfall for release through thousands of small springs.

The lower area, immediately upstream from the dam, is composed predominantly of barren, precipitous hills. Runoff from these hills, caused by the seasonal and often intense rainfall, produces many flash floods.

Water flowing past the damsite is not only regulated by the natural underground reservoirs, but also by numerous upstream reservoirs which have been constructed on the main stem and tributaries. As a result, the minimum flow of the Snake River at the Brownlee site is ideally suited for the generation of power. The minimum monthly average discharge recorded at the Oxbow gage was 7,000 cfs, while the mean discharge for the past 32 years was 17,000 cfs.

Site Description

The Brownlee damsite, about 70 miles from Weiser, Idaho, is located on the Snake River approximately 80 miles upstream from the confluence of the Snake and Salmon rivers. At the dam base line, the canyon is about 600 to 700 feet wide at river level and slopes up on both abutments at about 1 on 1; but the rock nose forming the left abutment just downstream from the base line has slopes as steep as 1/4 on 1. Rock outcrops and geological formations at the site indicate that bedrock lies a considerable distance beneath the valley floor.

Geologically, the project area is covered with thick beddings of miocene basaltic lava flows of the tertiary age. These beddings, as shown in Fig. 3, occur in massive layers which vary in thickness from 60 feet to more than 150 feet. The fine-grained basalt rock is dark grey when fresh, and becomes dark brown when exposed to air for any length of time. It contains crystals of feldspar in sizes from 1/4 inch to 1 inch in various beddings. By nature of its structure, the rock is hard, tough, and impermeable. Between the beds of lava flows, there are thin layers of tuff, varying in thickness from a few inches to a few feet.

At the damsite, the river cuts through five distinct beds which dip generally about 13° toward the west and form the easterly limb of a large syncline. The lowest of the beds is the most massive and extends across the river, underlying a major portion of the dam foundation.

The basalt rock layers contain natural joints which were formed during cooling of the lava flows and by movements of the crust of the earth. All beddings are closely jointed. The characteristic jointing and columnar formation was clearly evident on the left bank.

There were a few faults of small displacement in the area. These were considered to be inactive and therefore not a serious problem.

Of the few dikes at the damsite, the most noticeable was Dike No. 4, which cuts diagonally across the dam foundation and through the rock nose on the left abutment.

Project Description

Major features of the project are a rockfill dam, spillway, outlet works, and power facilities. (Fig. 2)

The dam has a crest length of 1,400 feet and a maximum height, as measured from bedrock to the top, of 400 feet. In section, the dam has a sloping impervious core flanked by sand and gravel filter zones and outer zones of rock. (Fig. 4)

The spillway, which is located in the rock nose of the left abutment, consists of an approach channel, a concrete control section, and a lined chute. Spillway discharges are regulated at the control section by four radial crest gates.

The outlet works consist of three gate-controlled openings through the control section of the spillway. Releases from the outlet works discharge into the spillway chute.

The power facilities include an intake channel, a power intake structure, individual steel penstocks in tunnels through the right abutment, and an outdoor type powerhouse. An initial installation of four units will be increased to six in the future, with an ultimate capacity of 540,000 kilowatts.

Foundation Explorations

The program of foundation explorations, conducted during preliminary and final stages of design, served to determine final design decisions. Of a total of 32 core-drill holes, the first eleven provided the data used in the selection of type of dam. Nine of the eleven were drilled in the river channel and two in the right abutment. There was a high percentage of recovery of all cores except those taken from tuff layers between the beddings and at breccia zones. Water tests of the holes indicated that only normal foundation grouting would be required.

The overburden was of considerable depth in the river channel with a maximum of 110 feet. It was generally well consolidated, and consisted of sand, gravel, cobbles, and small boulders. A deposit of sand and silt was found at the downstream end of a gravel bar near the left abutment.

Samples from some of the original holes indicated the presence of fine sand. If a rockfill dam were constructed at this site, seepage through the

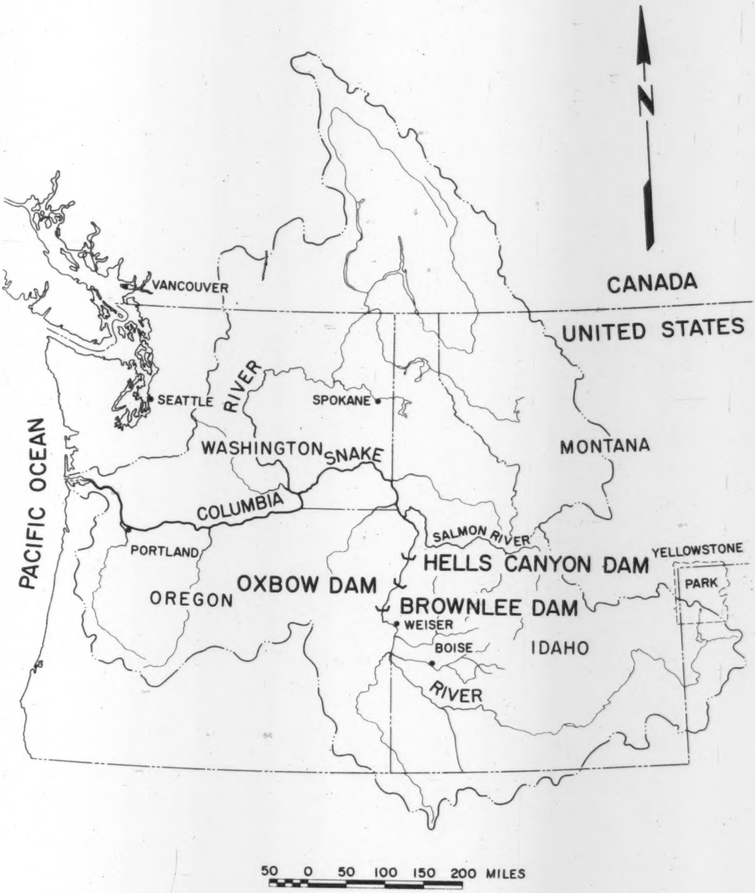

N
CANADA
UNITED STATES
VANCOUVER
SEATTLE
RIVER
SPOKANE
PACIFIC OCEAN
WASHINGTON
SNAKE
MONTANA
COLUMBIA
PORTLAND
SALMON RIVER
HELLS CANYON DAM
YELLOWSTONE
PARK
OXBOW DAM
OREGON
BROWNLEE DAM
WEISER
BOISE
IDAHO
RIVER
50 0 50 100 150 200 MILES

FIG. 1.—BROWNLEE PROJECT, LOCATION MAP

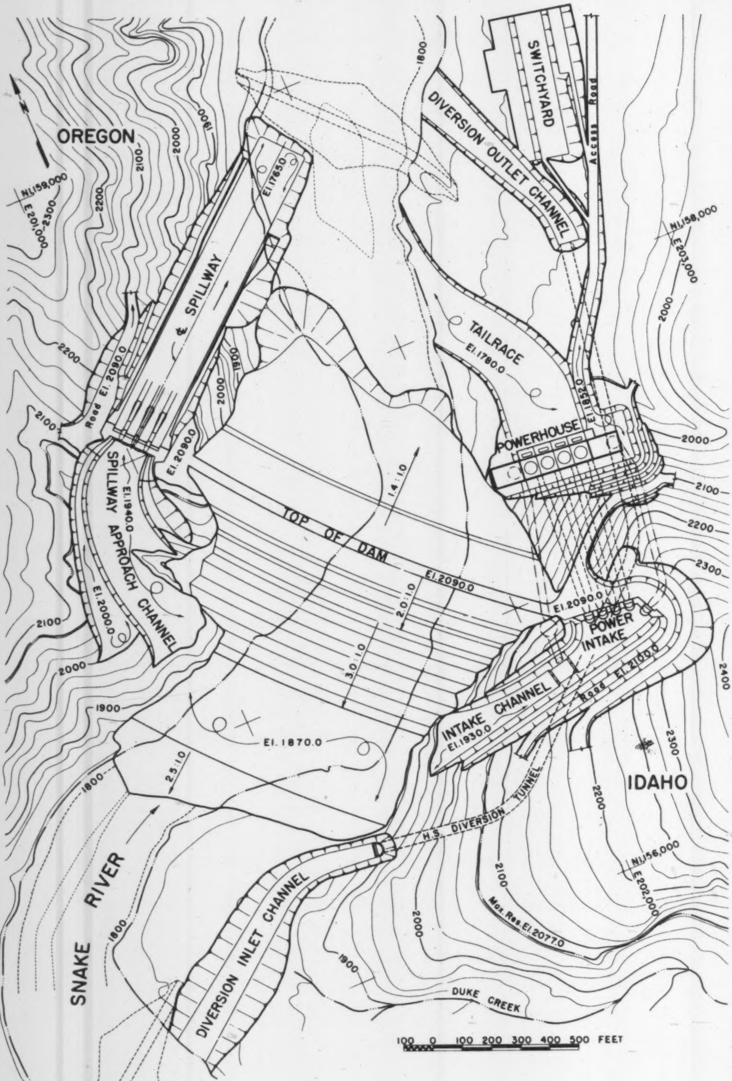
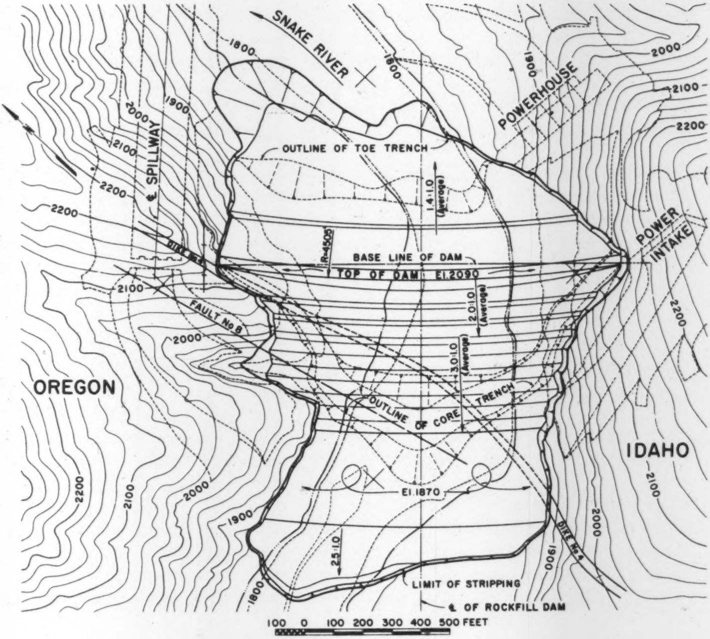
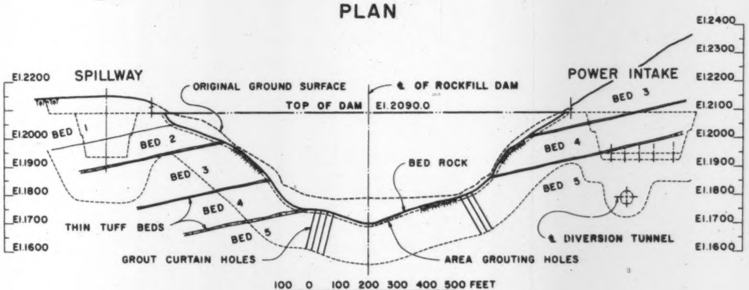

N
OREGON
NL159,000
E201,000
2300
2200
2100
2000
1900
1800
SWITCHYARD
Access Road
DIVERSION OUTLET CHANNEL
El. 1765.0
℄ SPILLWAY
Road El. 2090.0
TAILRACE
El. 1780.0
NL158,000
E203,000
POWERHOUSE
El. 1852.0
El. 2080.0
SPILLWAY APPROACH CHANNEL
El. 1940.0
El. 2000.0
TOP OF DAM
El. 2090.0
1.4:1.0
2.0:1.0
3.0:1.0
El. 1870.0
2.5:1.0
El. 2090.0
POWER INTAKE
INTAKE CHANNEL
El. 1930.0
Road El. 2100.0
2400
IDAHO
H.S. DIVERSION TUNNEL
NL156,000
E202,000
Max. Res. El. 2077.0
SNAKE RIVER
DIVERSION INLET CHANNEL
DUKE CREEK
100 0 100 200 300 400 500 FEET

FIG. 2.—BROWNLEE PROJECT, GENERAL PLAN


SNAKE RIVER
POWERHOUSE
℄ SPILLWAY
OUTLINE OF TOE TRENCH
1.4:1.0 (Average)
R=4505
BASE LINE OF DAM
TOP OF DAM El.2090
POWER INTAKE
DIKE No.4
FAULT No.8
2.0:1.0 (Average)
3.0:1.0 (Average)
OREGON
OUTLINE OF CORE TRENCH
IDAHO
El.1870
2.5:1.0
LIMIT OF STRIPPING
℄ OF ROCKFILL DAM
1800
1900
2000
2100
2200
100 0 100 200 300 400 500 FEET

PLAN


El.2400
El.2300
El.2200
El.2100
El.2000
El.1900
El.1800
El.1700
El.1600
SPILLWAY
ORIGINAL GROUND SURFACE
℄ OF ROCKFILL DAM
POWER INTAKE
TOP OF DAM El.2090.0
BED 1
BED 2
BED 3
BED 4
BED 5
BED ROCK
THIN TUFF BEDS
GROUT CURTAIN HOLES
AREA GROUTING HOLES
℄ DIVERSION TUNNEL
100 0 100 200 300 400 500 FEET

PROJECTED PROFILE ON ℄ CORE TRENCH

FIG. 3.—BROWNLEE PROJECT, ROCKFILL DAM

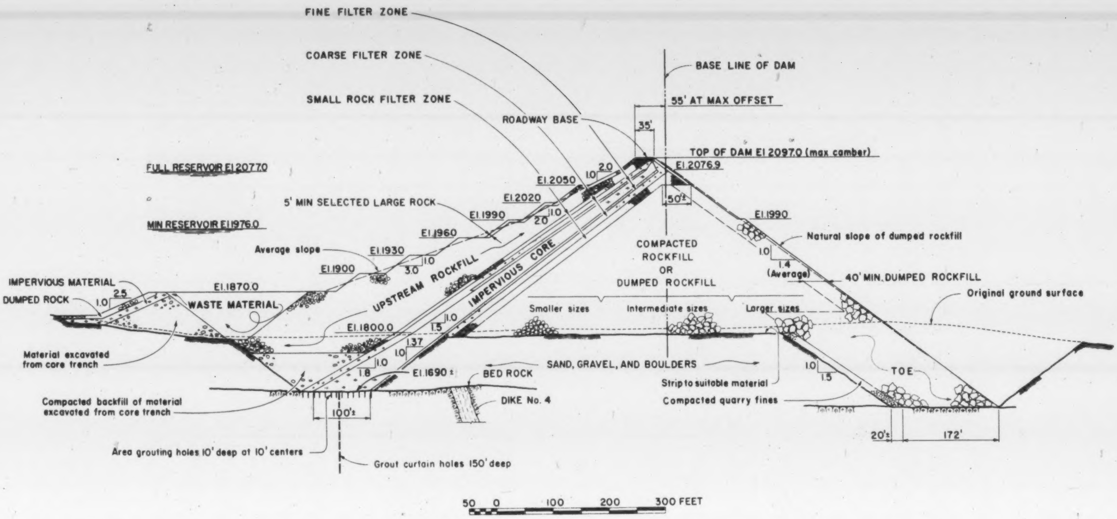

FINE FILTER ZONE
COARSE FILTER ZONE
SMALL ROCK FILTER ZONE
ROADWAY BASE
BASE LINE OF DAM
55' AT MAX OFFSET
35'
TOP OF DAM El 2097.0 (max camber)
El.2076.9
50'±
FULL RESERVOIR El 2077.0
MIN RESERVOIR El 1976.0
El.2050
1.0
2.0
El.2020
El.1990
5' MIN SELECTED LARGE ROCK
El.1960
Average slope
El.1930
3.0
1.0
El.1900
IMPERVIOUS MATERIAL
El.1870.0
DUMPED ROCK
1.0
2.5
WASTE MATERIAL
UPSTREAM ROCKFILL
IMPERVIOUS CORE
El.1800.0
1.5
1.37
1.0
1.8
El.1690±
BED ROCK
DIKE No. 4
Material excavated from core trench
Compacted backfill of material excavated from core trench
100'±
Area grouting holes 10' deep at 10' centers
Grout curtain holes 150' deep
COMPACTED ROCKFILL OR DUMPED ROCKFILL
Smaller sizes
Intermediate sizes
Larger sizes
SAND, GRAVEL, AND BOULDERS
Strip to suitable material
Compacted quarry fines
El.1990
Natural slope of dumped rockfill
1.0
1.4 (Average)
40' MIN. DUMPED ROCKFILL
Original ground surface
TOE
1.0
1.5
20'±
172'
50 0 100 200 300 FEET

FIG. 4.—BROWNLEE PROJECT, ROCKFILL DAM, MAXIMUM SECTION

dam and foundation could cause piping in the sand or liquefaction of the fine sand upon being saturated and subjected to load. Therefore, additional holes were drilled near those preliminary holes which had indicated the existence of fine sand. The additional holes revealed that the fine sand layers were limited in extent and thickness, and therefore not dangerous.

Rock samples obtained from the different bedding zones were carefully studied to determine the probable size of rock to be expected from normal excavation practices. Special attention was directed to the dip of the beddings in arranging the unusual deep excavation at the power intake, powerhouse, and the spillway.

Artesian water was encountered in all holes except the two near the left abutment downstream of Dike No. 4. In two holes on the right abutment, water stood at 30 and 58 feet above the river water surface. The higher water level was farther from the river, indicating that artesian water was percolating down the dip from the east limb of the syncline.

Selection of Type of Dam

The preliminary investigations and foundation explorations revealed that the rock formation was structurally sound and would present no major problems.

A concrete dam would be more expensive than a fill-type dam. The hundred feet of concrete dam below the present river bed at the deepest point would nearly double the cost of such a structure.

There is no large deposit of soil suitable for an earthfill dam near the damsite. There are, however, shallow patches of clay to be found in small deposits in the vicinity. The quantity of this material would be sufficient for the small amount of material required for the thin impervious diaphragm in a rockfill dam. Rock excavated at the site would be suitable for the dam. In addition, project cost could be reduced, by excavating all the rock required in the embankment from other features of the project. Increasing the length of the channel to the power intake structure reduces the penstock lengths, thus simplifying regulation of the units without increasing the dam embankment cost. Based on the above considerations and further detail studies of the elements of cost, the rockfill dam was selected as the most economical type to build at this site.

Investigation of Construction Materials

Following selection of the type of dam to be constructed, a soils laboratory was established at the site and a construction materials investigation program was initiated. Before the field soils laboratory was completely equipped to perform all tests, the soils laboratory of Washington State College was retained to make independent tests and to check the field laboratory work.

All the basic material components of the dam, such as the impervious core, filters and rock were thoroughly investigated before start of construction.

Impervious Materials

Extensive explorations in the vicinity of the damsite eliminated all but two potential sources of satisfactory impervious materials. Development of one of these sources would require construction of a 2-1/2-mile road on a six percent downgrade to the dam, while the other source would require processing of the material for removal of interspersed rock and gravel. Comparative cost studies indicated that development of the source requiring construction of the haul road would be the more economical.

Test results of typical samples from the selected site revealed the characteristics shown in Table 1.

TABLE 1.—IMPERVIOUS CORE MATERIAL TEST RESULTS

	B1 148	B1 149	B1 151	In Place Fill Material Test #1
Specific Gravity	2.78	2.90		2.85
Max. Dry Density* (lb/cu ft)	100.1	104.5	119.3	110.0
Optimum Moisture (%)	23.7	21.5	14.6	19.6
Liquid Limit	53.0	31.5	32.6	30.6
Plasticity Index	27.0	8.0	16.4	8.5
Permeability (cm/sec)	3.7×10^{-7}			2.1×10^{-5}
at Rel. Density (%)				
Cohesion (lb/sq ft)	3,400	2,450		
$\phi°$	15	19		16.7
Mech. Analysis** (% passing #200 sieve)	90	39	54	64

* Modified AASHO

** Mechanical analysis curves are shown in Fig. 5.

As indicated by the test results, the impervious material has a low friction angle, but very high cohesive strength. In addition, the material has the property of being exceedingly plastic, a characteristic considered to be very desirable for a thin core material.

The natural moisture content of the material varies from 20 to 25%, slightly greater than optimum.

Filter Material

Satisfactory sources for fine filter materials in their natural state were not found in the Brownlee area. The best material available was in a large sand and gravel deposit located about three miles upstream from the site, on a bench approximately 100 feet above the river. Under a layer of top soil several feet thick, the material varied in composition from fine sand to large gravel. As may be seen from the mechanical analysis shown in Fig. 6, the material lacked sufficient amounts of fine and medium sands to meet the requirements for fine filter. Therefore, a processing plant was erected to

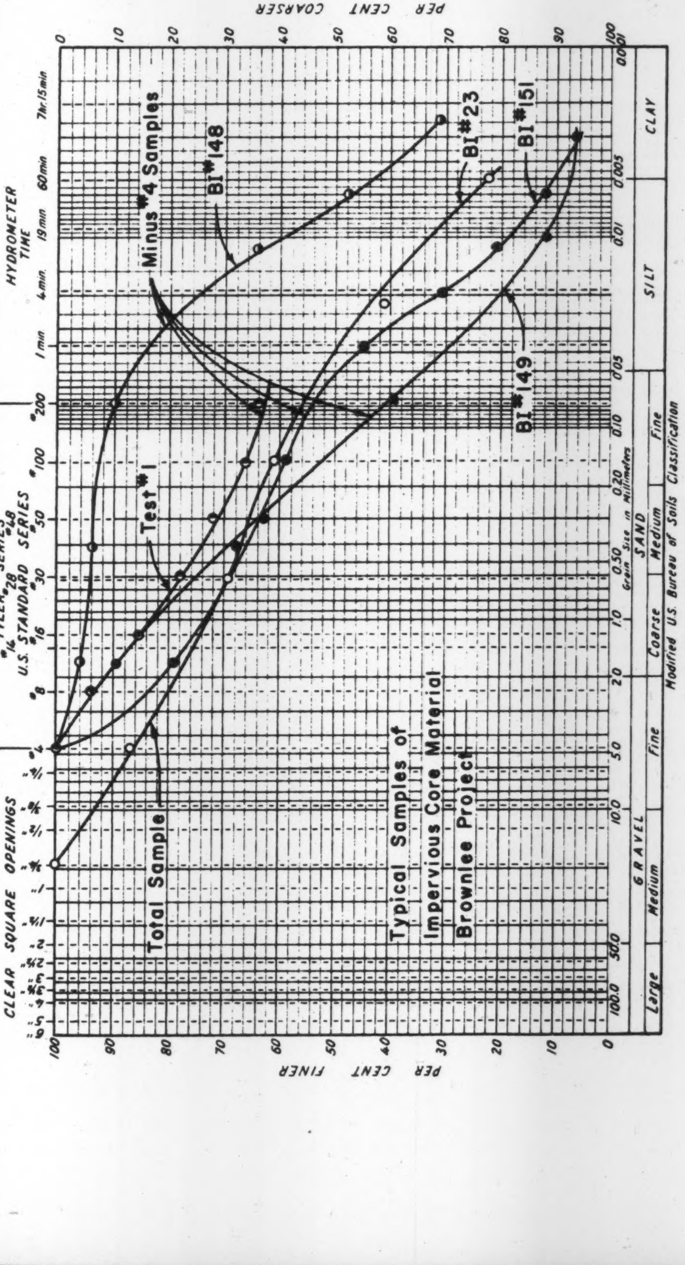


FIG. 5.—IMPERVIOUS CORE MATERIAL, MECHANICAL ANALYSIS CURVES

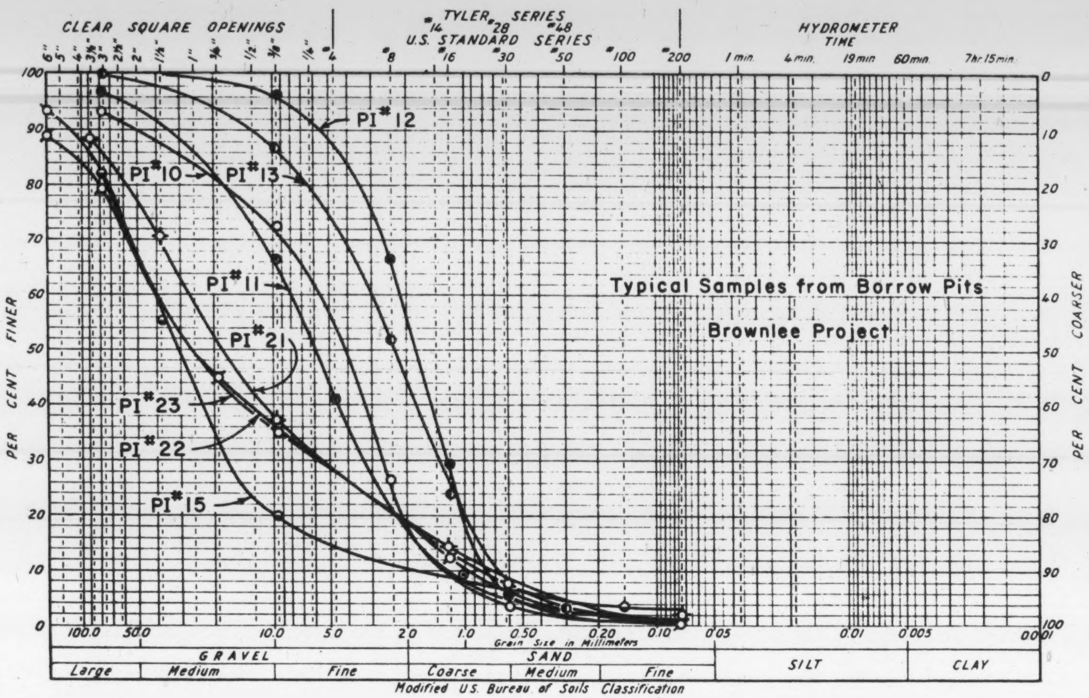


FIG. 6.—NATURAL FILTER MATERIAL, MECHANICAL ANALYSIS CURVES

screen out coarse material larger than 1-1/2 inches, and to blend the remaining material with fine river sand. This resulted in a satisfactory grain-size distribution. Mechanical analysis of the resultant material is shown in Fig. 7.

A "Wash Test" was used in the field laboratory to determine the ability of the fine filter material to retain the impervious material. At the start, water rushed through a wedge-shaped slot in the impervious material and into the fine filter compacted in the bottom of the test cylinder. Movement of the impervious materials gradually formed a seal which was 90% effective in twenty-four hours. By carefully removing the filter material in layers after the test, the amount and the depth to which the impervious material migrated into the filter was determined.

The naturally-deposited, fine filter material from the borrow area was mixed with the large-size gravel screened from the processed fine filter, to form the coarse filter material required.

Small-sized rock, ranging from 3 to 10 inches, was used as a third filter zone. This rock was obtained by selective loading from normal excavation.

Rockfill Material

Rock obtained from excavations made for the main structures constituted the major portion of the fill. Geological investigation had revealed that, with the exception of the weathered surface rock, all the rock from the excavations would be suitable for dam construction. The excavated rock sizes vary with the method of excavation, amount of explosive used, and the geological structure of the bedrock. For design requirements, rock for sluiced rockfill should be as large as possible, whereas for satisfactory construction, the rock should be of sizes that can be economically handled by available equipment. Trial excavations were made to determine the probable size of rock obtainable from the different areas of excavation.

The specific gravity of the solid basalt rock was found to be relatively high, due to the iron content, varying from 2.8 to 2.9. The natural angle of repose of the material, determined from measurements of the rockfill slopes at both abutments in the early stages of construction, varied from 1.35 to 1.40 horizontal to 1.00 vertical.

Final Design

Final design of the dam was governed by the criteria as listed below:

1. Arrangement of the basic project layout for the most effective use of topographic and geologic conditions.
2. Sufficient design flexibility to permit modifications required by changes in foundation conditions or material properties which might occur as construction progressed.
3. Fullest possible utilization of readily available construction materials.
4. Complete analyses of the main embankment for stability and expected settlement.

Project Arrangement

Arrangement of the project features was governed primarily by the topographic prominence of the rock nose which forms the left abutment, and by

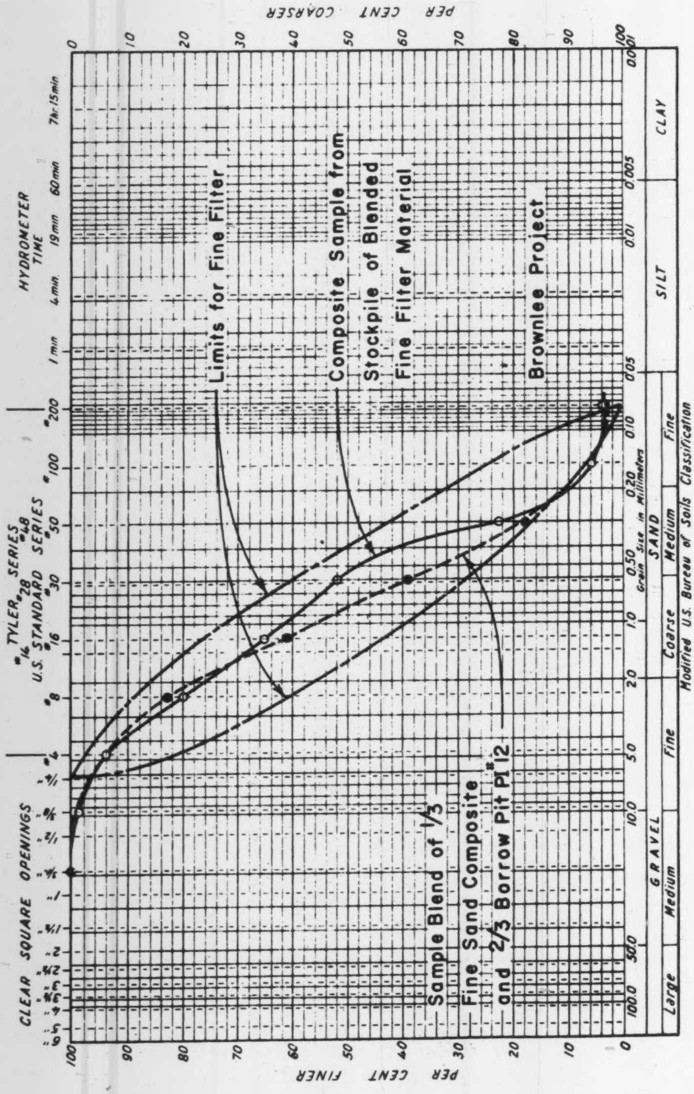


FIG. 7.—FINE FILTER MATERIAL, MECHANICAL ANALYSIS CURVES

the soundness of the rock in the right abutment. The rock nose provided an ideal location for a spillway, while the massive rock forming the right abutment was well suited for boring of the power and diversion tunnels.

Normally, the economic limit of excavation of project features is much less at other projects than at the Brownlee site. In this instance, excavated rock was usable in the fill; therefore, excavation was limited only by the amount required for the fill. This allowed considerable freedom in selecting locations and alignments for the project features. An outstanding example was the location of the power intake structure and the powerhouse. The selected location resulted in a cut over 450 feet deep on the intake side, and 420 feet deep on the powerhouse side of the ridge formed by the excavation. If the rock had not been required in the dam, cuts of this depth would not have been economically feasible. The only additional cost resulting from such arrangement was the provision of drains and grouting to stabilize the slopes of the ridge. A reduction in length of the power tunnels effected a considerable economy by reducing penstock and tunnel costs and eliminating the necessity for costly surge tanks.

Actual location of the dam was also influenced by the presence of Dike No. 4, which was considered suitable for treatment as a cutoff under the impervious core. The impervious core trench was located to follow this dike for as great a distance as possible.

Utilizing the rock nose on the left abutment required particular care in aligning the core for satisfactory abutment contact. To satisfy basic design requirements, the abutment slopes had to be free from sharp breaks or overhangs. The contact area of the sloping core was arranged to be normal to the core zone in a horizontal plane and to have as a flat a slope as economically feasible. In addition, it was not allowed to wrap around the nose. Careful studies and a model at the site were made to determine the abutment trimming which would satisfy these requirements with a minimum amount of excavation.

Flexibility

Providing flexibility of design was necessary to meet conditions arising during construction. The results of foundation explorations and material investigations indicated that either an inclined core or a vertical core would be suitable. The inclined core was selected because this type of arrangement provided flexibility in scheduling the dam construction. The major portion of the dam, the downstream rockfill, could be placed as soon as the rock from required excavation became available without stockpiling or delaying the construction of other features. Using an inclined core would also permit the placement of main rockfill ahead of the time-consuming construction of the core and filter zones which had to be scheduled according to the diversion program.

The embankment design should also allow the different sizes of rock to be placed at the appropriate portions of the embankment, with larger rock dumped near the outside faces and smaller rock near the core area. If necessary, the small rock could be placed with compaction in the central portion of the dam.

The extent of the filter zones below the riverbed was decided upon after completion of the core trench excavation, thus achieving the maximum economy of filter material.

Utilizing Available Material

Almost any excavated rock from the thick basalt beds at the site would have resulted in satisfactory material for the rockfill portion of the embankment. The selection of zone thicknesses, slopes, and method of placement was based on the estimated gradation and yield from the various excavations. The three portions of the embankment designed to utilize the rock material were: downstream toe, downstream rockfill and upstream rockfill.

Although hydraulic model tests indicated that no apparent downstream erosion would occur, the fact that about 100 feet of natural river-deposited material underlies the rockfill embankment was considered to be sufficient cause, to provide special toe protection. To accomplish this, a toe trench, excavated to bedrock and backfilled with selected large rock, was designed. The rock zone in the toe was to have a minimum section 172 feet thick, with a 20-foot filter blanket against the riverbed overburden.

The downstream rockfill was designed for two types of construction: one compacted rockfill, and the other dumped rockfill. The compacted rockfill design utilizes the smaller rocks obtained from excavation. The dumped rockfill utilizing the larger rocks, was designed with an average downstream slope of 1.4 horizontal to 1 vertical, the flattest slope obtained during field test investigations.

The upstream rockfill has an average upstream slope of 2 on 1 above Elevation 1990 feet, and 3 on 1 below. A wide berm below Elevation 1870 was constructed in front of the dam with material excavated from the core trench and with waste material from other excavations. This berm has a dual purpose: to serve as a cofferdam, and to stabilize the foundation upon which the upstream rockfill is resting. To avoid re-dressing costs, the upstream rockfill was built on natural slopes with intermediate berms every 30 feet.

In general, filter zone thicknesses, due to the high cost of processing the materials, were designed to be as thin as possible while still functioning properly. The minimum practical thicknesses, however, were often limited by construction placement methods.

Filter zones, classified as small rock filter, coarse filter, and fine filter were placed on both downstream and upstream sides of the core. The materials vary in size from 10 inches to 200 mesh, with smooth transition of gradation from rockfill to the impervious core. The downstream small rock rock filter zone extends from overburden to top of the dam, and was designed to have a minimum thickness of 15 feet. The coarse filter zone on the downstream side, designed to be 17 feet thick, was terminated at the junction with the overburden since the natural river material from that elevation down to bedrock was considered to be an equivalent substitute. The downstream fine filter zone extends from bedrock to top of the dam and has a five-foot thickness, except at the abutments where the thickness was increased to 10 feet.

The upstream small rock filter zone was designed to be nine feet thick and to cover the upstream coarse filter down to the elevation of the natural overburden material. The corase filter zone has a minimum thickness of 13 feet on the upstream side and extends down to bedrock, although the portion below the fine filter is considered only to be a buffer zone for the core material. The fine filter zone was designed to be five feet thick across the entire width, extending down to Elevation 1870 feet.

The impervious core, which extends from bedrock to the top of the dam, was designed with a downstream slope of 1.37 on 1, and an upstream slope of

1.5 on 1 above Elevation 1800 feet, and 1.8 on 1 below. The width of the contact surface between the impervious core and bedrock at Elevation 1690 feet is about 100 feet, which is considered to be conservatively sufficient to safely withstand the 387 feet of head. The width of the core at Elevation 2076.9 feet, the control elevation of the core, is 20.8 feet, and the thickness normal to the average slope at the same elevation is approximately 12 feet. The maximum amount of seepage through the core is expected to be less than one cubic foot per second.

Stability Analysis and Settlement

Stability analyses of the main dam embankment were made for both the upstream and downstream slopes. Potential failure plane on the upstream side of the embankment was considered to be along the impervious core, which is the normal assumption for an inclined core rockfill dam. Using the slideblock method, the analysis showed a minimum factor of safety of 1.38 for the most critical condition, which would occur during initial filling, when the reservoir water surface reaches Elevation 1930 feet.

There is normally little doubt about the stability of the downstream slope of a rockfill dam of ordinary height built on sound rock foundation. At Brownlee, however, due to the overburden, the stability of the downstream slope was analyzed along Elevation 1750 feet, the contact surface between the overburden material and the rockfill. With an internal friction angle of 28 degrees, the minimum factor of safety was found to be 1.53 for the critical condition, which was with the tailwater at Elevation 1827 feet.

The dam is slightly curved in an upstream direction on a 4505-foot radius. The maximum offset from the base line is 55 feet. Settlement will occur both downward and in a downstream direction. Due to the curve and the valley cross section, the impervious core upon settling will be squeezed, thus eliminating the possibility of cracks due to stretching.

Based upon past experience, a well-constructed rockfill dam could be expected to have a vertical settlement of less than one percent of its height. However, for appearance as well as for settlement, a maximum camber of seven feet was provided.

There is relatively little information in engineering literature about the settlement of rockfill dams, particularly those built on deep overburden. Previous settlement measurements have indicated that in the process of settling, some of the large rocks in the rockfill section have a tendency to rotate and thus cast doubt on the validity of the measurements. Monuments have been set in the Brownlee embankment to measure this rotation as well as the normal settlement of rockfill and core.

Construction

Careful consideration of equipment, time, and personnel, and a thorough economic analysis of the work processes involved, resulted in establishment of the following construction schedule:

November, 1955 to September, 1956

Mobilize
Build Access Road and Bridges
Excavate Powerhouse and Diversion Tunnel Headings

Drive Diversion Tunnel
Commence Rockfill

October, 1956 to February, 1957

Close Cofferdams and Divert River
Excavate Foundation Core Trench and Toe Trench
Grout Foundation
Construct Core and Filter Zones to Elevation 1800± Feet
Construct Downstream Rockfill to Elevation 1810± Feet in the River Channel Portion
Start Abutment Rockfill Placement

March, 1957 to June, 1957

Divert Over the Partially Completed Dam
Continue Placing of Rockfill from Both Abutments as Excavated Material Becomes Available

July, 1957 to February, 1958

Perform Necessary Clean-Up of the Embankment
Raise the Core and Fill to Where the Flood Will Pass Through Diversion Tunnel and Outlets

March, 1958 to June, 1958

Complete Embankment.

Wet Season of 1956

After mobilizing the necessary equipment, work progressed concurrently on several phases of construction. A bridge across the Snake River and access roads interconnecting the project features were rapidly completed. One of the roads connected the two portals of the diversion tunnel to allow use of one truck-mounted jumbo for drilling both tunnel headings.

The 2500 feet of 38 by 42-foot modified horseshoe section diversion tunnel was holed through in three months. Rock proved to be very good and only 300 feet of the tunnel required supports.

Rock from excavation for the powerhouse and muck from tunnel drilling was used to construct a section of the upstream berm of the embankment part way across the river. This preliminary embankment forced the river to flow on the Oregon side, where the water scoured out objectionable materials in the embankment foundation area.

Dry Season of 1956

When closure of the upstream cofferdam was completed and water diverted through the tunnel, the foundation area was unwatered and excavation of the core and toe trenches commenced on a 24-hour a day schedule. Unwatering was accomplished by two 20,000 gpm pumps, each installed in a pipe frame which could be picked up by a dragline and relocated as the excavation proceeded. Nearly 400,000 cubic yards of core trench excavation in the river channel were completed in less than three months. Equipment used to accomplish this excavation feat included two shovels of 3 and 3-1/2-cubic yard capacity, one 2-1/2-cubic yard dragline, one 5-1/2-yard Manitowoc dragline, and Euclids of 17-cubic yard capacity.

Foundation drilling and grouting was also accomplished in the same three-month period. The operations followed immediately after excavation of the core trench and proceeded from both abutments toward mid-river. Two systems of foundation grouting were used. One was a blanket grouting over the entire core trench area, and the other was a cutoff grout curtain.

The area grouting consisted of drilling holes on 10-foot centers a minimum of 10 feet deep. These holes were grouted under pressure of about 20 psi minimum. The grout "take" was very small in holes in the core trench area on the right portion of the river channel. Except near Dike No. 4, where a few holes took 12 to 34 cubic feet and one hole took 272 cubic feet, the amount of "take" was approximately that required to fill the drill holes. In the left portion of the river channel, the presence of seams and cracks in the foundation rock resulted in greater "take," varying from a few cubic feet to a maximum of 254 cubic feet in one hole with some loss to surface leaks.

Cutoff grout curtain holes were drilled to a depth of 75 feet on 20-foot centers with intermediate holes drilled to 150 feet. These holes were grouted under pressures up to a maximum of 300 psi. When the "take" was high, additional holes were drilled at 5 or 10-foot centers between the original holes. As in the area grouting, the right portion of the foundation took less grout than the left. The maximum "take" was 324.5 cubic feet in one 75-foot hole. A total of 58,961 linear feet of grout holes were drilled and grouted in the core trench with an average "take" of about 0.62 sacks of cement per linear foot of hole.

Core excavation revealed a fault near the center of the river channel. At the upstream end, the fault was 8 to 12 feet wide and filled with sand, gravel, and large boulders. The fault opening tapered to the thickness of a seam both in depth and in a downstream direction. Based on the grouting record, it was found that the fault was very tight.

While the foundation treatment was in progress, the borrow areas for impervious and filter materials were being prepared. One D-8 bulldozer was used to strip from two to five feet of soil overburden which was unsuitable for use in the core. Impervious clay core material was excavated from the borrow area by a 2-1/2-cubic yard shovel and loaded on bottom-dump Euclids. One shovel was able to efficiently supply loads to 15 Euclids of 17-cubic yard capacity. Round trip from borrow area to core area and back required about 45 to 60 minutes per truck. Shovel production varied from 100 to 150 cubic yards per actual working hour, but the average production per shift-hour, including moving, repair and idle time, ranged from 80 to 90 percent efficiency.

For the fine filter material, the main equipment used was the blending plant consisting of a 24" belt conveyor fed by bulldozers with fine river sand and pit-run filter material in a 30 - 70% proportion. A 1-1/2" screen separated the blended filter from the coarse material and the products were conveyed by belts to stock piles.

Immediately after foundation grouting in the core area was completed, placement of the clay core commenced. Artesian water entering the trench was encountered in lesser quantities than previously predicted since some of it was intercepted by the diversion tunnel, and some was deterred by specially drilled and grouted holes near the left abutment. However, since it was impossible to stop all the seepage to enable placement of the clay core on dry rock in the deepest portion of the core trench, which was the river fault area previously mentioned, a pile of clean gravels was used to fill the low spot with two 24-inch pipes, 8 to 10 feet long, embedded vertically. The gravel was capped with a concrete slab, and clay core material placed on top while seepage water was pumped through the 24-inch pipes. When the core reached a sufficient height, the gravel was grouted with cement through one of the pipes, forcing the water out through the other.

The impervious core was placed under rigid control, especially near the bedrock strata where it will be subjected to the highest water pressures. The "in-place" core material had a moisture content slightly higher than optimum. Tests of the material indicated a permeability less than 10 feet per year, and plasticity index higher than 10.

Initial compaction of the core against the rock foundation was effected by pneumatic hand tampers. When the working area became large enough, sheepsfoot rollers with specially designed wedge-shaped feet were used to compact the impervious material in about 6-inch layers. Usually, 12 to 16 passes were required to obtain the specified 90% modified AASHO optimum density.

The severe cold weather during the period might have caused progress to fall behind the tight construction schedule except for the fact that heat loss in the core material was small during hauling and placing operations. After hauling, spreading, and rolling the material, ice did not form in the completed layer, provided that the succeeding layer was immediately spread.

A typical temperature record in degrees Fahrenheit of one of the coldest days was reported as follows:

Air temperature	10
Clay temperature as hauled to the embankment	45
Temperature at surface of the compacted fill	31
Temperature at one-inch depth	32
Temperature at two inches depth	33
Temperature at three inches depth	34

If the operation was suspended too long, however, formation of needle ice in the clay could be observed. Therefore, clay chunks of suspicious nature were removed by hand when they could not readily be broken with a hand shovel.

More than 100,000 cubic yards of impervious material were placed in the core trench in less than two months. Progress on the core placement was stopped by the spring floods with the top at Elevation 1800 feet. The maximum daily placement progress (three shifts) amounted to more than 3,000 cubic yards of compacted impervious fill. (See Fig. 8).

Coarse filter materials were end or bottom-dumped, and fine filter materials were end-dumped against the coarse filters or the impervious core. The filters were compacted in the same operation, which resulted in a smooth transition from fine to coarse zones. Satisfactory compaction required at least four passes of the Euclids loaded with 17 cubic yards of material.

The toe trench was excavated to bedrock, and then backfilled with large rocks of which 50 percent were one-half cubic yard or larger in size.

Large selected rock, weighing one-half ton and over, will also be used to form a protective layer on the face of the upstream rockfill section.

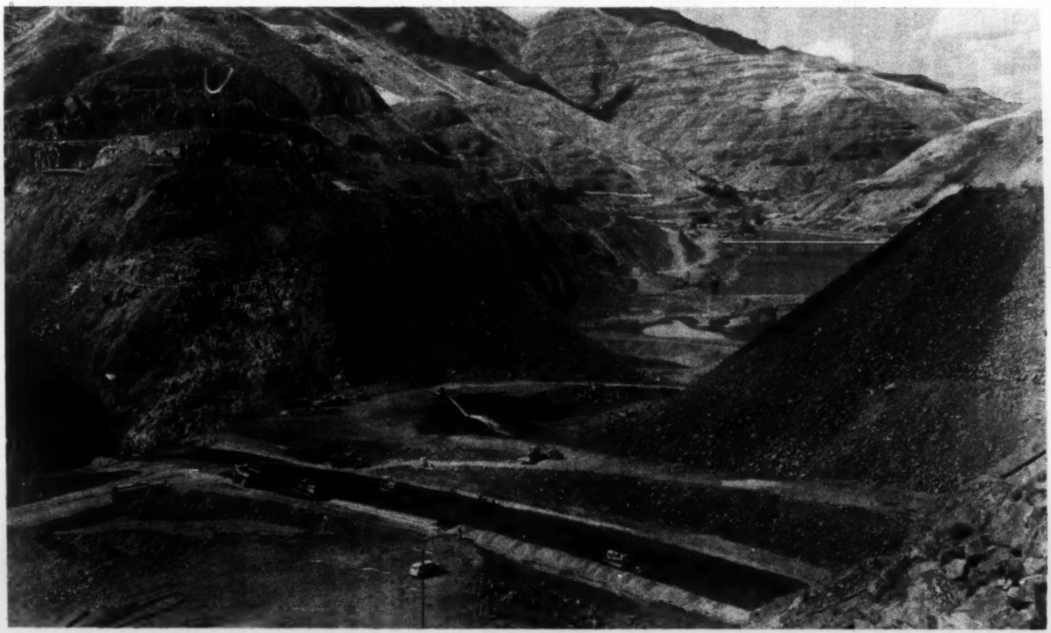


FIG. 8.—BROWNLEE DAM DURING CONSTRUCTION, VIEW LOOKING DOWNSTREAM SHOWING IMPERVIOUS CORE AND FILTER ZONES AND ROCKFILL

Wet Season of 1957

When the construction schedule was being formulated, one of the major considerations was river diversion. At the damsite, the low water season usually starts in late June or early July and lasts five to eight months. During this season, the flow varies from 8,000 to 20,000 cfs, and seldom exceeds 24,000 cfs. The first floods start as early as December, with normal high flood flows usually occurring in April or June with peak discharges from 50,000 to 70,000 cfs.

Computations disclosed that one 38-foot unlined horseshoe tunnel could pass the low water season flows, but two tunnels would be required to carry the high water season flows. Careful studies indicated that with a relatively small risk, diversion could be accomplished by constructing only one tunnel and allowing flood waters in excess of the capacity of the tunnel to flow over the partially completed fill. For the protection of the core area, it was planned to cover the core and filter areas with two to three feet of pit-run coarse filter materials and top with seven to eight feet of rockfill.

On February 24, 1957, however, intense rainfall on the barren hills immediately upstream from the site caused a flood peak to occur at the site very suddenly. All available men in camp were summoned to help move equipment out of danger in case the river overtopped the upstream cofferdam.

Before water reached the top of the upstream cofferdam, which was at Elevation 1835 feet, the road from the damsite to Robinette was seriously endangered by rising water. To save this road, the cofferdam was opened and water allowed to overflow the fill before the intended protection could be provided for the core. (Fig. 10) At the time of overtopping, the downstream
vided for the core. (Photograph 3) At the time of overtopping, the downstream rockfill was at Elevation 1850± feet near the abutments, and at Elevation 1809± feet for a width of about 250 feet in the central portion. The impervious core and filter zones in the core trench were approximately at Elevation 1800± feet.

The flood reached a peak of 70,000 cfs and an estimated 40,000 to 50,000 cfs flowed over the fill. There was approximately 20 feet of drop in the water surface in the 2,000-foot distance between cofferdams.

Following this first peak, the normal flood season set in and lasted about four months. During that time, the river flow ranged from 30,000 to 70,000 cubic feet per second, with the later flow occurring a second time on May 23, 1957.

Dry Season of 1957

By the middle of June, the river flow had receded to 20,000 cfs and the upstream cofferdam was again closed. Clean-up and inspection of the core zone, which began immediately after closure, revealed that the diversion scheme had been sound.

There was no apparent loss of rock in the main rockfill zones. The core area, which had been purposely kept lower than the rock area, was almost completely filled with mud and rocks to the elevation of the rockfill. Most of the material deposited in the core area apparently came from the upstream cofferdam.

Removal of this deposited material revealed that the original compacted impervious core had not been appreciably disturbed by the river waters. To ascertain if any internal change had occurred, a test pit five feet deep was excavated. Material removed from this pit showed no change in moisture content below the top few inches. Even in the top few inches, the increase was surprisingly slight. Only an insignificant amount of original core had to be removed before construction could be resumed. Diversion over the partially completed rockfill dam was considered to be a complete success.

Although the major portion of the rock in the embankment was placed during the 1957 dry season, placement actually proceeded throughout the entire construction period. Care was taken to meet the construction schedule and still maintain a balance between rock excavation and fill. Thus construction proceeded with no appreciable delays or stockpiling of rock. This record was maintained not only for the dry season of 1957, but for the entire construction period.

Along the right abutment, the first lift of rock was dumped at about Elevation 1900 feet. Embankment proceeded simultaneously toward the downstream toe and mid-river. As the trucks end-dumped their loads over the edge of the embankment, a monitor mounted on the outer end of a skid which stuck out over the edge of the embankment sluiced the rock down the slope using a 2-3/4 inch nozzle and water pressure ranging from 75 to 100 pounds per square inch. The monitor supplied about four times as much water by volume as the rock being sluiced.

On the Oregon side, rock placement began on the gravel bar away from the left abutment. Placement reached an elevation of 1850 feet and then progressed toward the abutment. By this means, a good contact was made between the sluiced rock and the steep abutment slopes. When the spillway excavation reached top production, another lift was started from this abutment at Elevation 1900 feet, and then a final lift at Elevation 2050 feet. Rock placement proceeded from both abutments, at equal elevations, gradually closing the gap between the slopes in the middle.

The amount of smaller-sized rock obtained from the spillway excavation was larger than had been expected. The excess 6-inch and smaller rocks were placed and compacted in the center portion of the dam. Analysis of the compacted small rocks indicated that stability of the fill was not impaired. In addition, the amount of settlement is expected to be no greater than in the sluiced rock portion. The small rocks were similar to the small-rock filter zone material and were placed and compacted in the same manner; that is, in 18-inch layers compacted by not less than three passes of 50-ton pneumatic rollers. After each lift was compacted, the surface was sluiced. All of the small rock was covered with not less than 40 feet of large sluiced rock at the downstream face of the dam.

Placement of the downstream rockfill progressed far ahead of core and upstream rockfill placement. (See Fig. 9)

Excavated rock which was considered unsuitable for use in the downstream rockfill because of small size or too high a percentage of fines, was placed in a portion of upstream rockfill zone. This area which was designed for use of this material did not require all the material and the remainder was placed in the upstream berm. The large rock portion of the upstream rockfill was sluiced in place by monitors.

Rockfill Production

During the peak rockfill operations, 6 monitors and 48 Euclids were in use. Maximum production was about 30,000 yards in two 9-hour shifts.

Practically all rockfill material was obtained from excavation for other features. Prior to June 1957, there was limited space for placement of the

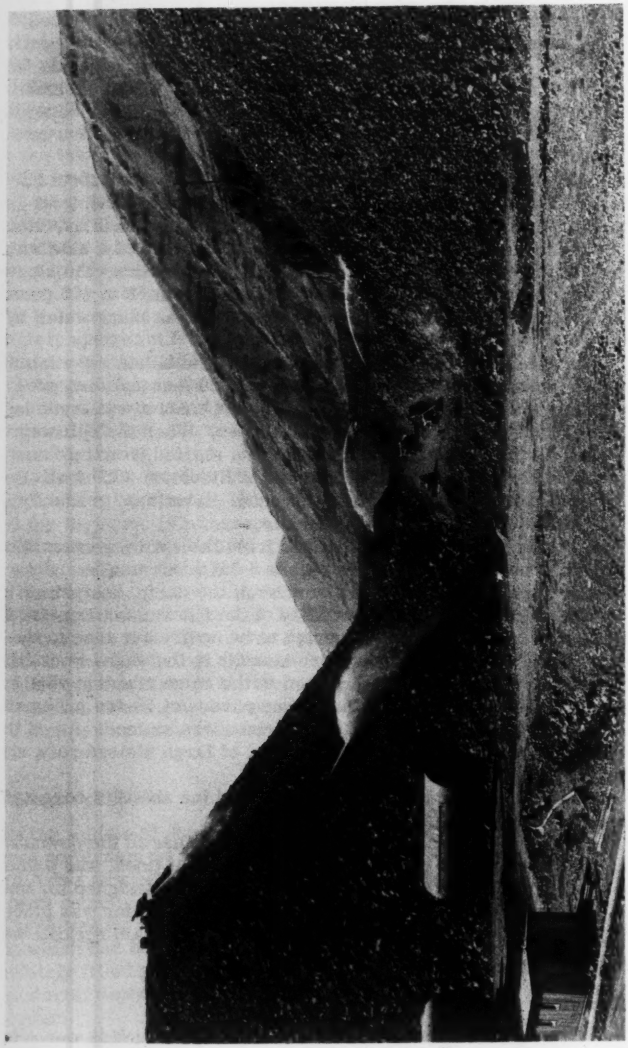


FIG. 9.—ROCKFILL CONSTRUCTION, VIEW LOOKING UPSTREAM SHOWING DUMPED ROCKFILL BEING PLACED AND SLUICED

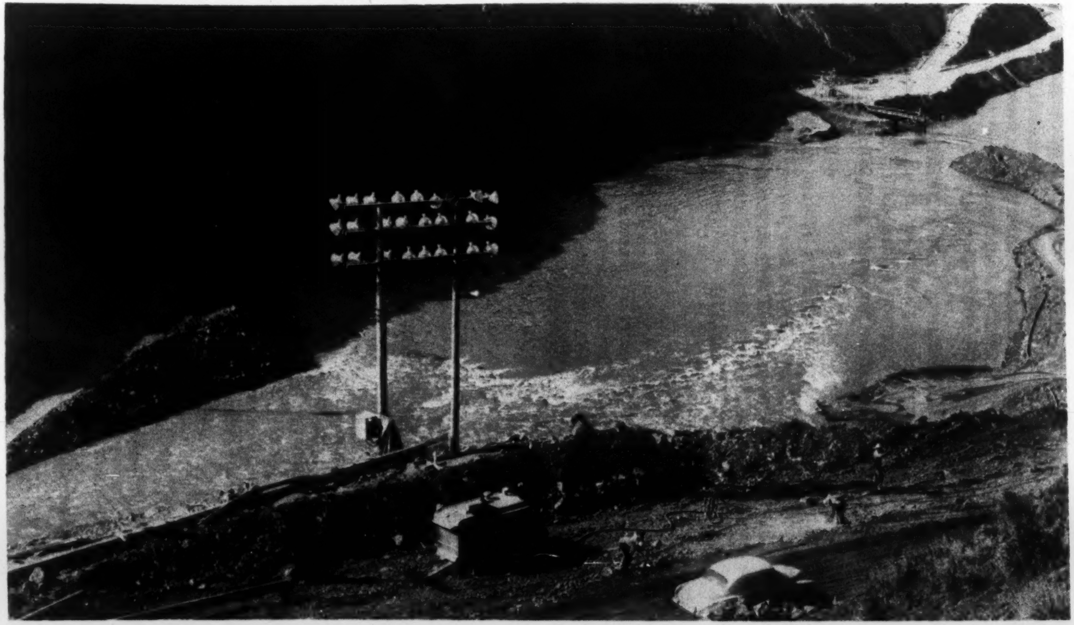


FIG. 10.—DIVERSION OF RIVER OVER PARTLY COMPLETED DAM, VIEW LOOKING DOWNSTREAM FROM THE IDAHO ABUTMENT SHOWING THE FLOOD CHANNEL THROUGH THE ROCKFILL

excavated rock in the dam. Total volume placed to that date was about 2,300,000 cubic yards, of which 800,000 cubic yards were from powerhouse and tailrace excavations, 700,000 from power intake, and 800,000 cubic yards from spillway. After clean-up from river immersion was accomplished, construction of both the core and rockfill proceeded at top speed.

The major embankment quantities placed, in cubic yards, were as follows:

Upstream cofferdam	150,000
Downstream cofferdam	150,000
Upstream rockfill	1,582,000
Downstream rockfill	3,976,000
Filter zones	468,000
Impervious core	449,000
Upstream berm to El. 1870 feet	706,000
Total Fill	7,481,000

Rock excavation was accomplished with drills of the percussion type and bit sizes varying from 2-1/2 inches to 4 inches. Lifts varied from 15 to 25 feet with holes from 5 feet to 10 feet apart on centers. The production averages were as follows:

	Drilling	Powder Rate
Powerhouse	0.72 L.F./cy	0.48 lb/cy
Power Intake	0.70 L.F./cy	0.61 lb/cy
Spillway	0.66 L.F./cy	0.55 lb/cy

The amount of rock excavation in cubic yards obtained from required excavation was as follows:

Powerhouse	477,000
Tailrace	440,000
Spillway	1,690,000
Power Intake	1,400,000
Core Trench	377,000
Toe Trench	403,000
Diversion Tunnel	156,000
Diversion Inlet Channel	143,000
Diversion Outlet Channel	183,000
Total Excavation	5,269,000

CONCLUSIONS

1. Brownlee Rockfill Dam will be completed in July 1958. Unit cost for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago. Improvements in earth-and-rock-hauling equipment in recent years, combined with modern construction methods, have made rockfill-type dams much more competitive with other types.

2. The large quantities of rock required in the embankment made the heavy excavation at other features of the project feasible and resulted in substantial savings of the individual feature costs. Careful site adaption, arrangement of the project and scheduling of operations were required to achieve the overall project economy.

3. Rockfill dams may be safely supported on consolidated riverbed materials of sand, gravel, and boulders provided foundation explorations have been performed to ascertain that there are no extensive silt of fine sand layers. Careful analysis of rockfill embankments should be made to satisfy the stability requirement.

4. When normal rock excavations provide a considerable quantity of small rocks, economics can be effected without loss of structural strength by placing them in the center of the embankment with sufficient compaction.

5. Passing flood water over a partially completed rockfill dam is entirely feasible. Before adopting such a diversion scheme, detailed hydraulic analyses should be prepared to determine probable flow characteristics over the embankment.

DISCUSSION

GEORGE K. LEONARD,[2] F. ASCE.—Although the paper gives no hint of what the final unit costs of rock excavation and rock fill will be it is evident, with almost all of the required rock fill being furnished by the required excavation, that the combined cost of the excavation and the rock in place will be unusually low. Also, the estimated kilowatt cost of $175 for the initial installed capacity would be pleasing to the owners.

The project planning was thorough and complete. Full advantage was taken of all features of site topography and geology, the availability of rock and impervious core material, and knowledge of anticipated weather conditions which would affect work schedules and diversion schemes.

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[2] Cons. Engr., Knoxville, Tenn., Formerly, Chf. Engr., TVA, Knoxville, Tenn.

The plan for economical diversion during construction was especially noteworthy. There must be considered risks taken in all diversion plans for it is generally uneconomical to design cofferdams and diversion channels to handle maximum possible flows. Plans made at Brownlee involving flow over the partially completed fill worked out nicely although some fast work by the construction forces was necessary. The planners, designers, and constructers all did a praiseworthy job in completing this fine dam on a fast schedule.

F. L. LAWTON,[3] **F. ASCE.**—The author has established as criteria applied to the final design of the Brownlee dam those considerations which enter into the economic design of any rockfill dam. Two features are of particular note in this connection: the thin impervious core necessitated by the absence of large volumes of suitable soil, and the deep overburden in the river channel. Another aspect of major importance in securing optimum economy at Brownlee, in common with many other rockfill dams, was the achievement of a close balance between rock excavation and rockfill requirements.

Effective utilization of geological features usually results in superior economy. This would appear to have been fully realized in the use of the rock nose in the left abutment, of the overburden in the river bed and of dyke No. 4 for a portion of the core trench.

The method used to facilitate placement of the clay core in the deepest portion of the core trench, despite seepage, was ingenious. It is a method analogous to the use of a concrete pad on a jagged, steeply-dipping bedded formation in order to use tractor-drawn compaction equipment in the relatively-restricted area in the lower part of a river channel, as at Kenney dam,(4) and also achieve better grout distribution.

It is rather difficult to fully appreciate how ". . . special drilled and grouted holes near the left abutment" deterred some of the artesian water from entering the core trench in view of the geological profile along the centre line of the core trench. Perhaps the author could amplify his observations on this point.

The overtopping of the incomplete dam during the period from February 24, 1957, to some time about mid-June through a gap about 250 feet wide at approximately Elevation 1809 provided an extremely valuable experience since an estimated 40,000 to 50,000 cfs flowed over the fill. The author's observations on the behaviour of the rockfill and impervious core would have been much more valuable if the effect on the fine and coarse filter zones had been noted. Information on the size of the rockfill would also be useful.

Under conclusions, the author states the "unit cost for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago". This is such an important factor in the choice of type of dam for a given location, it is to be hoped the author can justify the conclusion by giving supporting figures.

In connection with the construction of the Brownlee dam on the consolidated alluvials (sand, gravel and boulders), it is worth noting the left-hand bank section of the Bersimis dam(5) is built on a maximum thickness of about 300 feet of a succession of till and varved clay and silt strata, of glacial origin. This material is obviously much finer than that underlying the Brownlee dam. Both Brownlee and Bersimis dams are striking examples of one of the attractive economic features of the rockfill dam, its application to apparently difficult, if not impossible, dam sites where the properties of well-designed thin impervious cores, filters and blankets are fully utilized.

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[3] Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

[4] "The Kenney Dam," by Harry Jomini, The Engineering Journal, November, 1954, Vol. 37, No. 11, p. 1386-1397.

The author's conclusion on the feasibility of passing flood water over a partially completed rockfill dam is an important one, as such a procedure can be utilized to minimize diversion costs, where probable flood flows to be handled during construction can be reasonably closely evaluated and construction schedules permit overtopping.

HUAI-YUN HSU,[6] M. ASCE.—The stability of Brownlee rockfill dam was briefly discussed in Mr. Mundal's paper. The detail analysis which follows may be of interest to design engineers.

Upstream Slope

In rockfill dams, generally, the upstream rockfill has better shear strength than the filter zones and the filter zones better than the impervious core. Unless the upstream rockfill material contains too high a percentage of fines or clay, the weakest shear plane should be in the impervious core. In Brownlee, the critical surface selected for stability study is along ABCD as shown in Fig. 11. The vertical line CE divides the wedge into two parts—Block ABCE sliding down along the potential failure plane BC and Block ECD resisting the failure by its shearing strength along the base CD. The position of line CD is so chosen that a minimum factor of safety will be obtained. The exact location of AB is immaterial, as the forces acting on it are relatively small and, therefore, neglected.

Taking block ABCE as a free body, besides the resisting force R (equal to $W_1 \tan \phi_R$), we have the forces T and N, tangent and normal to line CD, to keep the weight W_2 in equilibrium. T and N are solved by the force polygon.

Since the potential shearing strength, P, along the line CD is equal to $N \tan \phi_c + CL$, the factor of safety against failure along CD is P over T. However, in computing the factor of safety for the entire surface ABCD, the value of R should be reduced by the factor of safety before entering the force polygon.

With an internal friction angle of 40 degrees for the upstream rockfill, the factor of safety for the upstream slope at Brownlee was found to be 1.56 for

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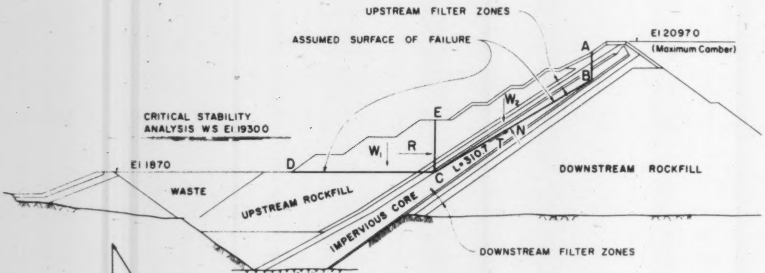
5. "Rockfill Dams: The Bersimis Sloping Core Dams," by F. W. Patterson and W. H. MacDonald, Transactions, ASCE, Vol. 125, Part II, 1960.

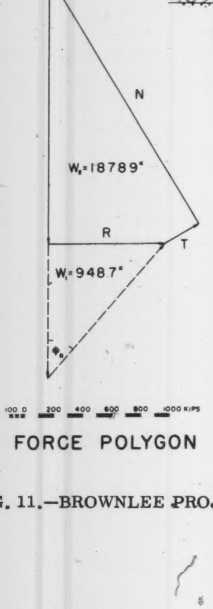
[6] Project Engr., International Engr. Co., Inc., San Francisco, Calif.

SOIL CHARACTERISTICS

PROPERTY	UNIT	ROCKFILL	IMPERVIOUS CORE
MOIST WEIGHT	P.C.F.	115	110
SATURATED WEIGHT	P.C.F.	134.4	117.4
SUBMERGED WEIGHT	P.C.F.	72	55
COHESION, C	P.S.F.	—	1000
ANGLE OF FRICTION, φ°	Deg.	40°	15°


UPSTREAM FILTER ZONES
ASSUMED SURFACE OF FAILURE
El 2097.0
(Maximum Camber)
A
B
W_2
E
N
T
CRITICAL STABILITY
ANALYSIS W.S. El. 1930.0
W_1
R
L=310.7
El 1870
D
C
DOWNSTREAM ROCKFILL
WASTE
UPSTREAM ROCKFILL
IMPERVIOUS CORE
DOWNSTREAM FILTER ZONES

MAXIMUM SECTION


N
W_2=1878.9^K
R
T
W_1=948.7^K
ϕ_R
100 0 200 400 600 800 1000 KIPS

FORCE POLYGON

ASSUMED CONDITIONS INITIAL FILLING OF RESERVOIR

WITH CRITICAL STABILITY ANALYSIS W.S. AT El 1930.0

ASSUMED SURFACE OF FAILURE ABCD

ASSUMED FACTOR OF SAFETY OF ROCK (FS_R)	1.0	1.5	2.0
W_1 =	948.7^K	948.7^K	948.7^K
$R = \frac{W_1 \tan \Phi_R}{FS_R}$	796.0^K	530.7^K	398.0^K
$P = N \tan \Phi_c + CL$			
N (Scaled)	2,024^K	1,889^K	1,820^K
$N \tan \Phi_c$	542.3^K	506.2^K	487.7^K
CL	310.7^K	310.7^K	310.7^K
TOTAL P	853.0^K	816.9^K	798.4^K
T (Scaled)	272^K	500^K	614^K
FACTOR OF SAFETY OF CORE $FS_c = \frac{P}{T}$	3.14	1.63	1.30
FACTOR OF SAFETY OF FAILURE SURFACE ABCD		1.56	

CALCULATION OF FACTOR OF SAFETY

FIG. 11.—BROWNLEE PROJECT, STABILITY ANALYSIS, UPSTREAM SLOPE

the most critical condition of initial filling of the reservoir with water surface at El. 1930. Using 28 degrees for ϕ_R results in a factor of safety of 1.38 for the same conditions.

Downstream Slope

Brownlee rockfill dam is underlain by more than 100 feet of riverbed sand and gravel. Although dense and compacted, the riverbed foundation material has a smaller angle of internal friction than the main rockfill for the dam. The stability of the downstream slope was, therefore, analyzed along the contact surface, between the foundation and the dam, at different elevations across the river. The example shown in Fig. 12 is for El. 1700.

On any vertical plane AB above the plane of investigation, there is a force P, acting on the block ABC from the left. The forces acting on the base BC are the vertical reaction and the horizontal shear resistance (equal to $(P_V + W) \tan \phi_f$). Therefore the factor of safety against shear failure on the contact surface is:

$$\text{F. S.} = \frac{(P_V + W) \tan \phi_t}{P_H} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which ϕ_f is the angle of internal friction of the foundation material.

The value and direction of the force P was determined by Engesser's graphical method for cohesionless material with the reservoir water load included. The minimum factor of safety on the contact surface at El. 1700 was found for point B at a distance of 280 feet from the downstream toe of the rockfill. The data and the result are as follows:

Critical tailwater elevation (max. T.W.)	El. 1827 feet
Angle of internal friction for rockfill, ϕ_R	40 degrees
Angle of internal friction for foundation material, ϕ_f	39 degrees
Moist weight of rockfill	115 lb/cu. ft.
Submerged weight of rockfill	72 lb/cu. ft.
P (scaled from Engesser's Envelope)	855 kips
P_V	515 kips
P_H	690 kips
W	2155 kips

$$\text{F. S.} = \frac{(515+2155) \tan 39^\circ}{690} = 3.1$$

Laboratory tests indicate that the values of the angle of internal friction used in the stability studies are conservative.

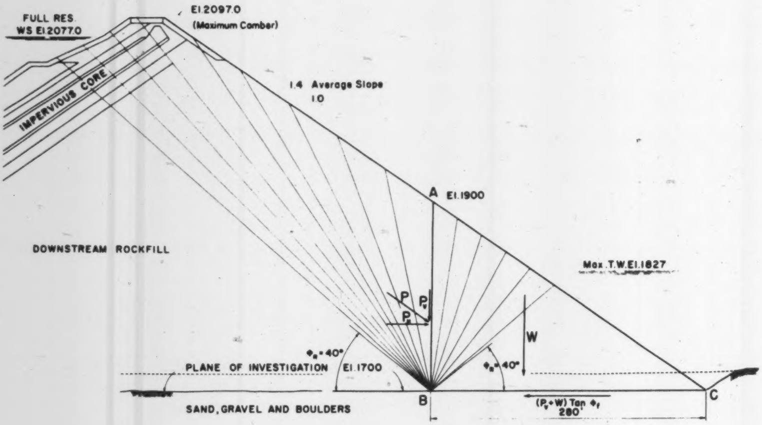
By utilizing Engesser's method of determining the value P, it is possible to determine the stresses at any point in the main rockfill.

$$\text{Vertical pressure, } P_v = \frac{d(W+P_V)}{dx} \quad \ldots\ldots\ldots\ldots \quad (2a)$$

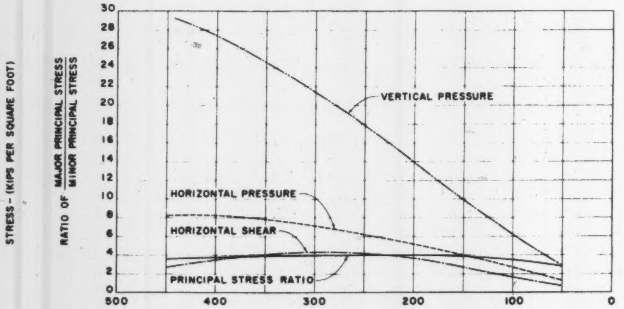
$$\text{Horizontal pressure, } P_h = \frac{dP_H}{dy} \quad \ldots\ldots\ldots\ldots \quad (2b)$$

$$\text{Horizontal shear, } \tau = \frac{dP_H}{dx} \quad \ldots\ldots\ldots\ldots \quad (2c)$$

where x and y are the horizontal and vertical axes. Graphical differentiation can be performed easily.


FULL RES.
WS El.2077.0
El.2097.0
(Maximum Camber)
IMPERVIOUS CORE
1.4 Average Slope
1.0
A
El.1900
DOWNSTREAM ROCKFILL
Max. T.W. El.1827
P
P_v
P_H
W
$\phi_R = 40°$
$\phi_R = 40°$
PLANE OF INVESTIGATION
El.1700
B
C
SAND, GRAVEL AND BOULDERS
$(P_v + W) \tan \phi_f$
280'

MAXIMUM SECTION



FOUNDATION STRESSES AT EL. 1700

FIG. 12.—BROWNLEE PROJECT, STABILITY ANALYSIS, DOWNSTREAM SLOPE

With vertical and horzontal stresses known, the major (σ_1) and minor (σ_3) principal stresses can be determined readily by Mohr's circle, or calculated by formulas. The stress distribution of the Brownlee dam foundation at El. 1700 is shown in Fig. 12 for an angle of internal friction of 40 degrees for the rockfill material.

For cohesionless material in a state of plastic equilibrium, the following relation holds:

$$\frac{\sigma_1}{\sigma_3} = \tan^2\left(45^\circ + \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Therefore, for the angle of internal friction of the foundation material at Brownlee equal to 39 degrees, the ratio of the principal stresses should not exceed 4.4 The actual maximum stress ratio on the contact surface at El. 1700 was 4.0 as shown on Fig. 12.

REIJO ITO.[7]—The writer has read Mr. Mundal's paper on Brownlee Dam with much interest as the type and scale of this dam are very similar to those of the Miboro Rockfill Dam now under construction by the Electric Power Development Co., Ltd. of Japan. Miboro Dam is a sloping core rockfill dam of 130 meter (426 feet) height (Fig. 13). Work began in 1957 and is scheduled to be completed in 1960. By briefly explaining the Miboro Dam construction, the writer wishes to give data to the readers which will become part of the records on high rockfill dam construction.

Geology

The bed rock in the dam site consists of quartz porphyry and granite porphyry. The bed rock of the right abutment is rather poor as talus deposits are found along the lower slope on the porphyritic rocks mentioned above and as cracks have developed in the bed rock itself. In the left abutment, quartz porphyry of relatively good quality is found exposed at several spots, but thick talus deposits cover the upstream and downstream sides of the abutment. Thickness of the deposit in the river bed is relatively thin and is less than 8 meters.

A conspicuous fault is found along the toe in the right abutment of the dam and is parallel with the river. The dip of this fault is about 70° NE, and the width of the portion coming in contact with the impervious core zone is about 10 meters, 2 meters of which is a well-compacted clay layer and the remainder consisting of crushed rock.

Selection of Type

A gravity type of concrete dam was first planned, but due to unfavorable geological conditions and especially owing to the right abutment fault, it was found that the volume of foundation excavation and concrete would have to be increased. Foundation treatments would also become more involved. Moreover, it was found that rock of good quality and materials suitable for an impervious core zone could be found in areas not very far from the dam site. For these reasons, a rockfill dam design was selected. A sloping core type

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[7] Chief of Construction, Miboro Dam Project, Electric Power Development Company of Japan, Miboro, Japan.

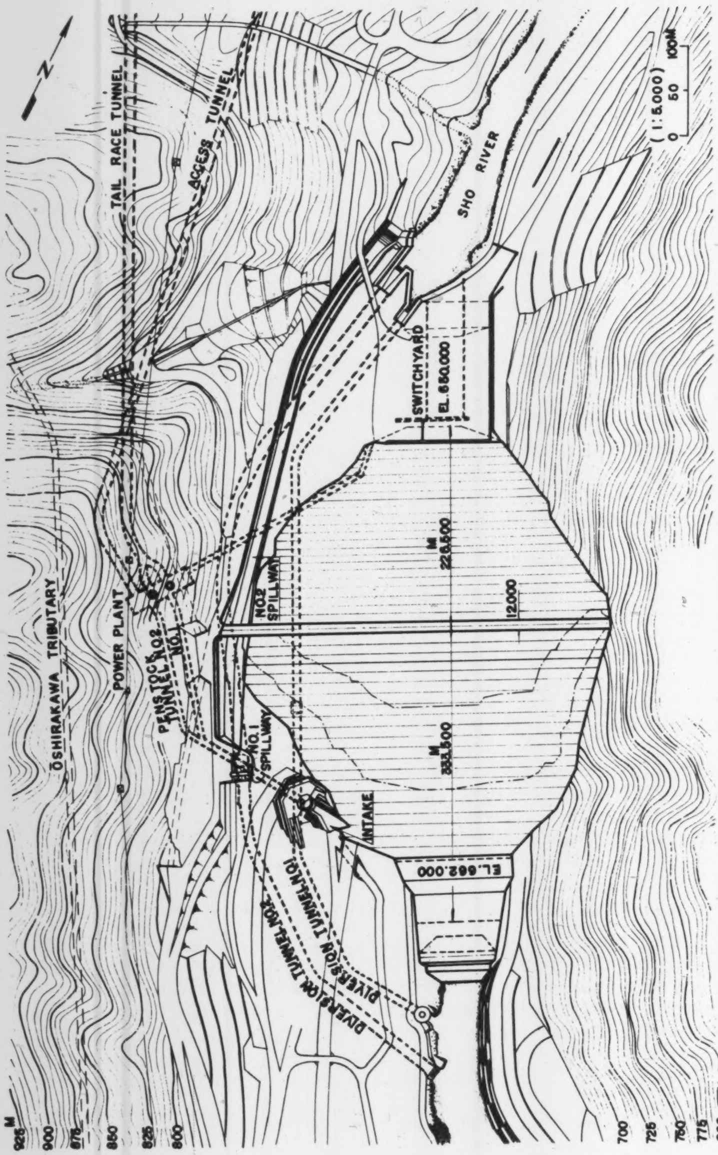
N
TAIL RACE TUNNEL
ACCESS TUNNEL
SHO RIVER
(1:5,000)
0
50
100M
SWITCHYARD
EL. 650.000
M
226.500
12.000
NO.2 SPILLWAY
POWER PLANT
PENSTOCK TUNNEL NO.2
NO.1
ŌSHIRAKAWA TRIBUTARY
NO.1 SPILLWAY
INTAKE
M
333.500
EL. 662.000
DIVERSION TUNNEL NO.1
DIVERSION TUNNEL NO.2
M
925
900
875
850
825
800
700
725
750
775
800

FIG. 13.—MIBORO DAM

of rockfill dam, as shown in Fig.14, was designed based on investigation results of materials found near the dam site.

Volume of the dam is 7,950,000 m^3 (10,400,000 cu. yd.) which is broken down as follows: main rockfill, 3,720,000 m^3 (4,880,000 cu. yd.); upstream rockfill, 1,760,000 m^3 (2,310,000 cu. yd.); impervious core, 1,630,000 m^3 (2,140,000 cu. yd.); and filter zones, 840,000 m^3 (1,100,000 cu. yd.).

Foundation Treatment

Excavation totaling 1,500,000 m^3 (1,960,000 cu. yd.) was completed during August 1958. Bed rock for the foundation of the core zone was completely exposed by removing top soil, sand and gravel layers. All soft and weathered portions of the bed rock were removed by hand excavation. In the upstream rockfill and main rockfill foundation areas it was first planned to remove silt, sand, gravel and disintegrated rock, which might contribute to the settlement of the dam. Actually, both abutments were excavated to the bed rock, while portions of the river bed where sand, gravel and rocks were more compact were left as the dam foundation without exposing the bed rock.

Grouting was accomplished from two concrete cutoffs, Fig. 14. There are three classes of holes for curtain grouting: "a"—10 m in depth, "b"—20 m to 30 m, "c" 30 m to 60 m. The packer grouting method was chiefly applied in the order of "c", "b", and "a". The maximum grout pressure was 15 kg/cm^2 (213 psi) and 2 to 3 kg/cm^2 (28 to 43 psi) near the surface. Brittle and weak spots encountered in the bed rock and in places where spring water was noticeable in the right abutment, blanket grouting was done to depths ranging from 7 to 12 meters using pressures from 2 to 3 kg/cm^2.

A vertical shaft was excavated near the crushed zone in the right abutment fault. In addition to grouting from four adits cutting the fault and connected to the shaft on different elevations, it is planned to backfill excavated portions with concrete where grouting alone is considered to be insufficient.

Rockfill

Material for the rockfill is obtained by means of coyote hole and large drill hole types of blasting in the Fukushimadani quarry which is located about 2 kilometer upstream from the dam site. Relatively large rocks are placed in the downstream and smaller sizes in the upstream portions of the main rockfill. In addition to the foregoing, muck of good quality produced from underground power plant and tunnel excavation is utilized for the upstream side of the main rockfill. Material for the upstream rockfill is mostly obtained from the Fukushimadani quarry and in addition oversize boulders produced at the filter materials plant from river gravel are also utilized.

In order to prevent segregation in sizes, the height of lifts of dumped rocks is limited to 4 meters in the upstream and 8 meters in the downstream portions of the main rockfill. A maximum lift height of 4 meters for the upstream rockfill zone is specified. All material placed in the main rockfill is sluiced and the amount of water applied is more than three times as much as the volume of the rock sluiced. The water pressure at the sluicing nozzles is approximately 5 kg/cm^2 (72 psi). No sluicing is required for the upstream rockfill.

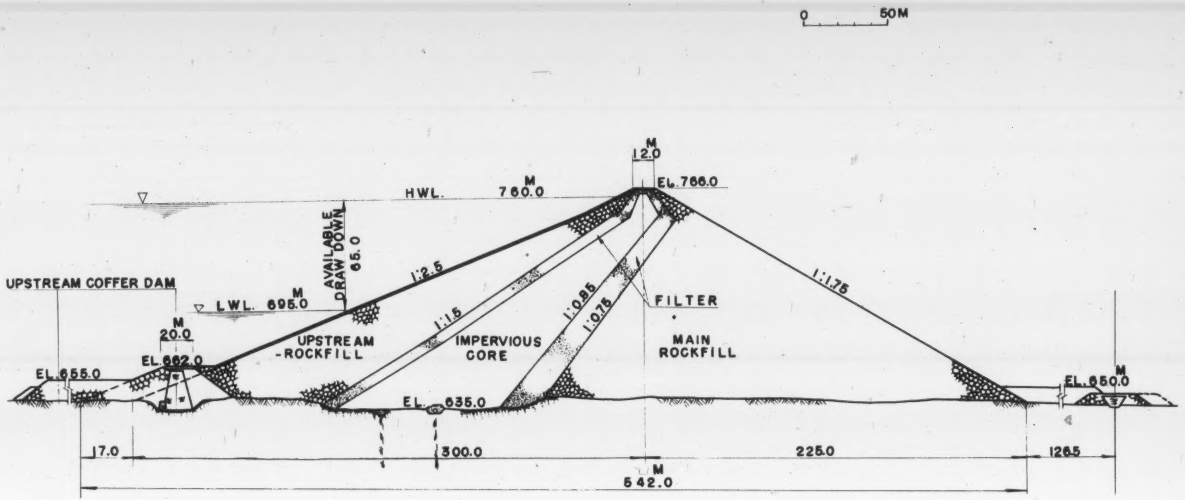
0
50M
M
12.0
EL. 766.0
HWL.
M
760.0
AVAILABLE
DRAW DOWN
65.0
1:2.5
UPSTREAM COFFER DAM
LWL. M 695.0
M
20.0
EL. 662.0
EL. 655.0
UPSTREAM
-ROCKFILL
1:1.5
IMPERVIOUS
CORE
1:0.85
1:0.75
FILTER
MAIN
ROCKFILL
1:1.75
M
EL. 650.0
EL. 635.0
17.0
300.0
225.0
126.5
M
542.0

FIG. 14.—TYPICAL SECTION OF MIRBORO DAM

Filter

Pit run sand and gravel from river deposits is screened at a special plant and all 200 mm minus material is used for the filter zones.

Impervious Core Materials

Due to heavy rainfall and high humidity in Japan, clayey materials having a suitable permeability coefficient as core material have in general a natural moisture content higher than the desired optimum. Miboro is situated in one of the heavy rainfall zones of Japan. Following the rainy season in June there may be several typhoons bringing heavy rains during the months of August and September. The job is shut down December through March due to considerable amounts of snow fall. Core materials which were found in the Miboro area similar to BI #148 and BI #151 at the Brownless Dam contained more than 60% and 30% of moisture respectively.

Under these unfavorable conditions, investigations were made covering wide areas extending 12 km upstream from the dam site in order to locate satisfactory core material pits. After making tests on moisture content, gradation, Atterberg limits, Proctor density, permeability coefficient, shearing strength and consolidation together with careful studies on samples obtained from various test pits, adits and auger holes, it was concluded that it would be impossible to obtain a large volume of satisfactory materials for the impervious core zone in a dry condition. As a result, a decision was made to adopt a policy of mixing two or more types of materials found in the Akimachi area which is located about 3 km upstream the dam. In this area the hill sides face the south and west with terraces along the lower slopes. As these terraces are wide and slope gently to the south, the whole area is favorably oriented to absorb the maximum heat from the sun effectively drying the material. In most portions of the hillsides, disintegrated granite (D.G.) is exposed when the relatively thin top soil is removed. In the gullies and lower slopes, clayey materials containing much detritus lie between the D.G. and the top soil. In the terraces of the lower slopes thick layers of clayey material containing much detritus is found under the top soil. As a result, it is much easier to prepare a satisfactory core material by first mixing these several types of materials in a stockpile within the area.

Gradation curves are shown on Fig. 15 and test results on representative samples obtained from the Akimachi area are tabulated in Table 2.

Test samples I and II are D.G. which contain very small amounts of fines and show no stickiness. Natural moisture content of the D.G. is lower than optimum while density and shearing strength are higher. The permeability coefficient is quite high. By adding some clay these materials become very suitable for an impervious core.

Test samples III and IV are clayey soil and although contain detritus, their fines are very sticky. The natural moisture content is higher than the optimum and the density is lower. Compaction of this type of material by rollers is very difficult and it cannot be used by itself. It is therefore used as a mixing agent with D.G. material in order to increase the imperviousness of the latter.

Mixing of two or more different kinds of materials such as mentioned above is successfully carried on by making stockpiles on the flatter slopes. In this method, there are several advantages as enumerated in the following:

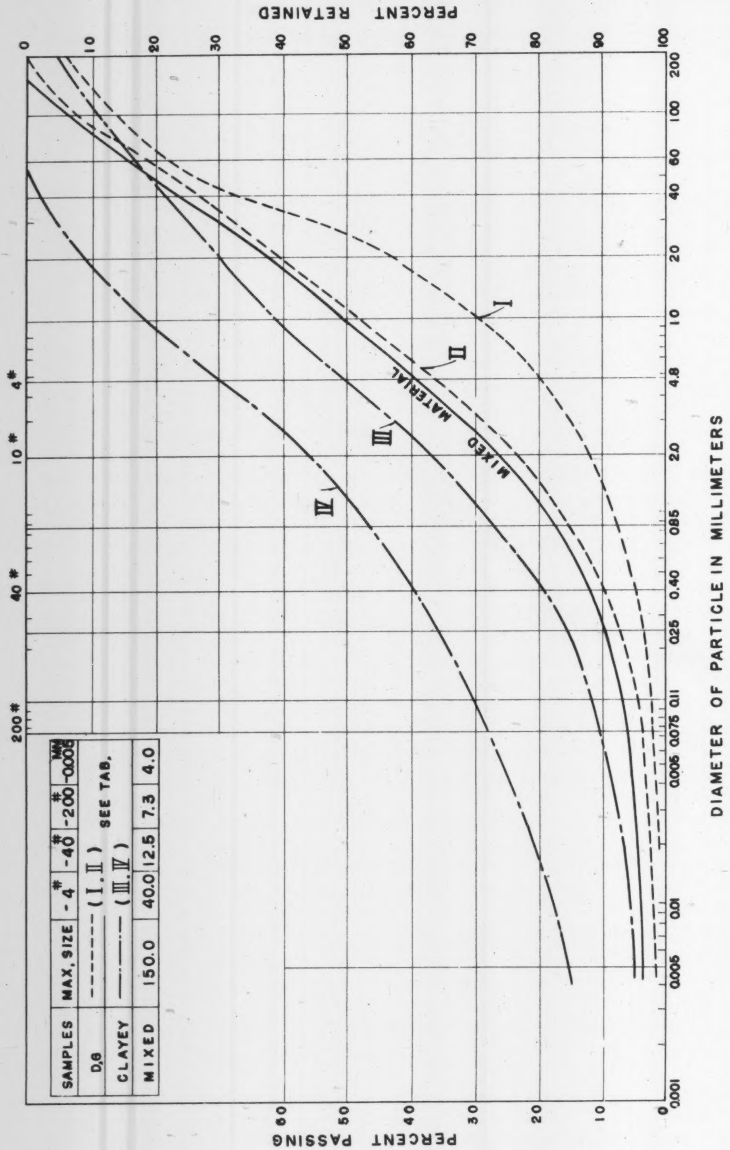

FIG. 15.—GRADATION CURVES OF CORE MATERIALS

TABLE 2.—TEST RESULTS ON EACH CORE MATERIAL

Classification of Core Material			Sample I GW	Sample II GW	Sample III GM	Sample IV SM - SC
Gradation	Max. size	(mm)	400.0	200.0	300.0	50.0
	-4 #	(%)	20.0	37.0	50.0	70.0
	-40 #	(%)	6.0	10.0	19.0	40.0
	-200 #	(%)	3.0	4.0	11.0	27.0
	-0.005 mm	(%)	1.5	1.5	5.0	16.0
Specific Gravity			2.69	2.70	2.64	2.65
Natural Moisture Content		(%)	7.0-12.0	7.0-14.0	15.0-21.0	17.0-30.0
Standard Compaction	Optimum Moisture Content	(%)	12.5	13.5	17.8	18.5
	Max. Dry Density	(t/m^3)	1.95	1.90	1.72	1.70
		(lb/cu. ft)	122	119	107	106
Atterberg Limits	Liquid Limit	(%)	31.0	34.0	39.0	40.0
	Plasticity Index	(%)	3.0	5.0	5.0	10.0
Shearing Strength	Moisture Content	(%)	12.5	13.0	17.8	17.5
	Dry Density	(t/m^3)	1.83	1.92	1.69	1.70
		(lb/cu ft)	114	120	106	106
	Cohesion	(kg/cm^2)	0.60	0.50	0.45	0.50
		(lb/sq in)	8.5	7.1	6.4	7.1
	Angle of Internal Friction	(Degree)	35°-00'	36°-00'	28°-40'	26°-00'
Consolidation Amount (Δ)	Moisture Content	(%)	12.8	13.2	18.2	19.0
	Dry Density	(t/m^3)	1.90	1.88	1.70	1.67
		(lb/cu ft)	119	118	106	104
Pore Pressure (P)	Load Δ	(%)	3.0	4.0	4.0	5.0
	10 kg/cm^3	P (kg/cm^2)	1.0	1.5	2.2	3.0
	142 psi	psi	14.2	21.3	31.2	42.6
Permeability Coefficient		(cm/sec)	1.5×10^{-5}	1.5×10^{-5}	6.5×10^{-6}	2.0×10^{-7}

(1) By placing the D.G., whose natural moisture content is low, after a rain or during the night, and by placing a thin clay layer on top during a dry day, the moisture content of clay can easily be lowered. Thus, it is possible to lower the moisture content of the mixed materials as a whole and, as a result, the number of core placement days can be increased considerably.

(2) Oversize rocks and boulders can be removed during the stockpiling operation.

(3) Different materials stocked in horizontal layers are excavated almost vertically by shovels so that the materials are mixed evenly and thoroughly during this operation.

(4) When placing, a uniform material of known quality is obtainable so that it is possible to construct a more uniform core zone. This also permits simplification of compaction work in the dam, and results in an increased rate of core placement. An example of properties of stockpile mixed materials is shown in Table 2.

TABLE 3.—PROPERTIES OF STOCKPILE MATERIALS

	Maximum Size	(mm)	150.0
Gradation	-4 #	(%)	40.0
	-40 #	(%)	12.5
	-200 #	(%)	7.3
	-0.005 mm	(%)	4.0
Specific Gravity			2.65
Natural Moisture Content		(%)	10.0 - 14.0
Standard Compaction	Optimum Moisture Content	(%)	14.0
	Max. Dry Density	(t/m^3)	1.87 (117 lb/c.f)
Shearing Strength	Cohesion	(kg/cm^2)	0.55 (7.8 psi)
	Angle Internal Friction	(Degree)	34° - 10'
Consolidation (Load 10kg/cm^2)	Consolidation	(%)	4.0
	Pore Pressure	(kg/cm^2)	1.6 (22.8 psi)
Permeability Coefficient		(cm/sec)	$< 1 \times 10^{-5}$

Twenty ton sheepfoot rollers are being used for core zone compaction. Of the 2,000,000 m^3 (2,620,000 cu. yd.) of all materials placed in July-December, 1958, 250,000 m^3 (326,000 cu. yd.) was core material. Of the 203 tests made for moisture content of core materials placed during this period, 80.2% of the total showed that compaction was made on the dry side of the optimum.

Mr. W. E. Collins acts as a consulting engineer in the design of the dam, while engineers of Guy F. Atkinson Company are giving technological assistance relative to its actual construction work.

G. H. CONDIT.[8]—An attempt has been made to follow the movement of Brownlee dam as closely as possible. It is unfortunate that construction methods do not make it possible to begin measurements soon enough to plot initial settlement and horizontal movement. Monuments for measurement purpose were not placed along the crest until all cable trench work and final dressing along the top was completed, so the possibility of damage to the monuments would be greatly reduced.

The position of the monuments installed is shown in Fig. 16. Monuments A, B and C are placed along an upstream bench at Elev. 2050; these monuments are used for settlement measurement only. Readings were begun on these points four months before those along the top and continued for three months after the others were begun which permits a correlation with those along the top. Since that time they have been under the water surface. Monuments along the crest of the dam consist of a minimum of one cy of concrete with an imbedded brass cap. The "F" settlement rods are supported on a bearing plate on the top of the clay core. They are set inside a 16 inch pipe and supported at intervals by a spider to provide an accurate reading for settlement but not for horizontal movement.

Total lateral and vertical displacement for monuments and vertical displacement for settlement rods on September 15, 1959 is shown on Fig. 17. It will be noted that the maximum displacement for the two types of readings does not occur at the same station on the dam. Maximum total lateral movement measured was at A4 which is in area of deepest toe trench excavation and at an area where the finer river sands and gravels existed in the river bed. The greatest vertical settlement has taken place at Station 13 + 13 which was the deepest area in the core trench.

For Fig. 18 settlement readings for approximate quarter points on the dam and the monuments on the 2050 bench have been plotted against the headwater elevation. It can be seen that there has been a distinct relation between the settlement rates and the headwater elevation. The rate of settlement lagged the headwater change rate by approximately one month. Lateral movement of A4, which had the greatest change, showed the same relation. Table 4 is a listing of the monthly changes in horizontal movement. In all cases, except one, there was a definite upstream shift following the lowering of the water. Table 5 is a similar tabulation for settlement.

No axial movement readings were taken until September 2, 1959. As would be expected there is a definite movement axially from each end of the dam

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[8] Resident Engr., Idaho Power Company.

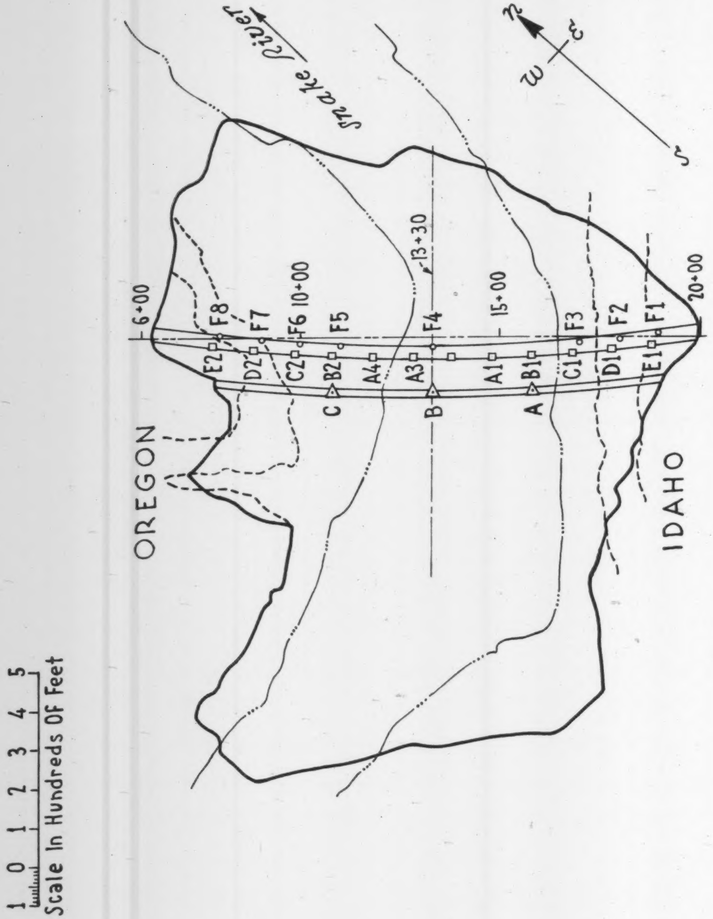
1 0 1 2 3 4 5
Scale In Hundreds Of Feet
OREGON
IDAHO
Snake River
N
E
W
S
6+00
10+00
13+30
15+00
20+00
F8
F7
F6
F5
F4
F3
F2
F1
E2
D2
C2
B2
A4
A3
A1
B1
C1
D1
E1
C
B
A

FIG. 16.—MONUMENT LOCATION PLAN

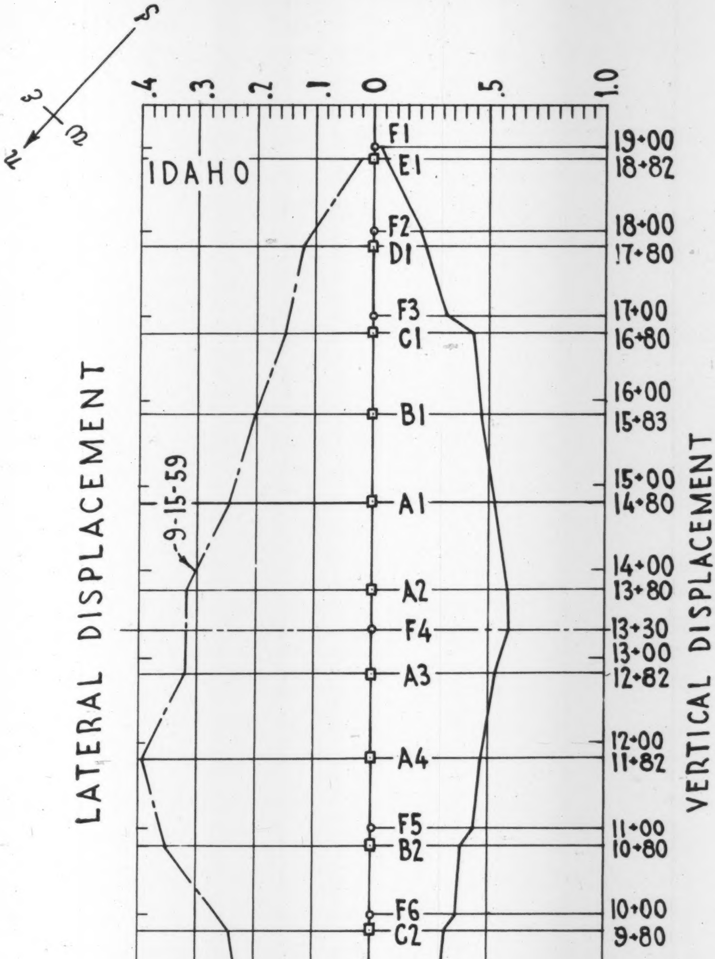

FIG. 17.—DISPLACEMENTS

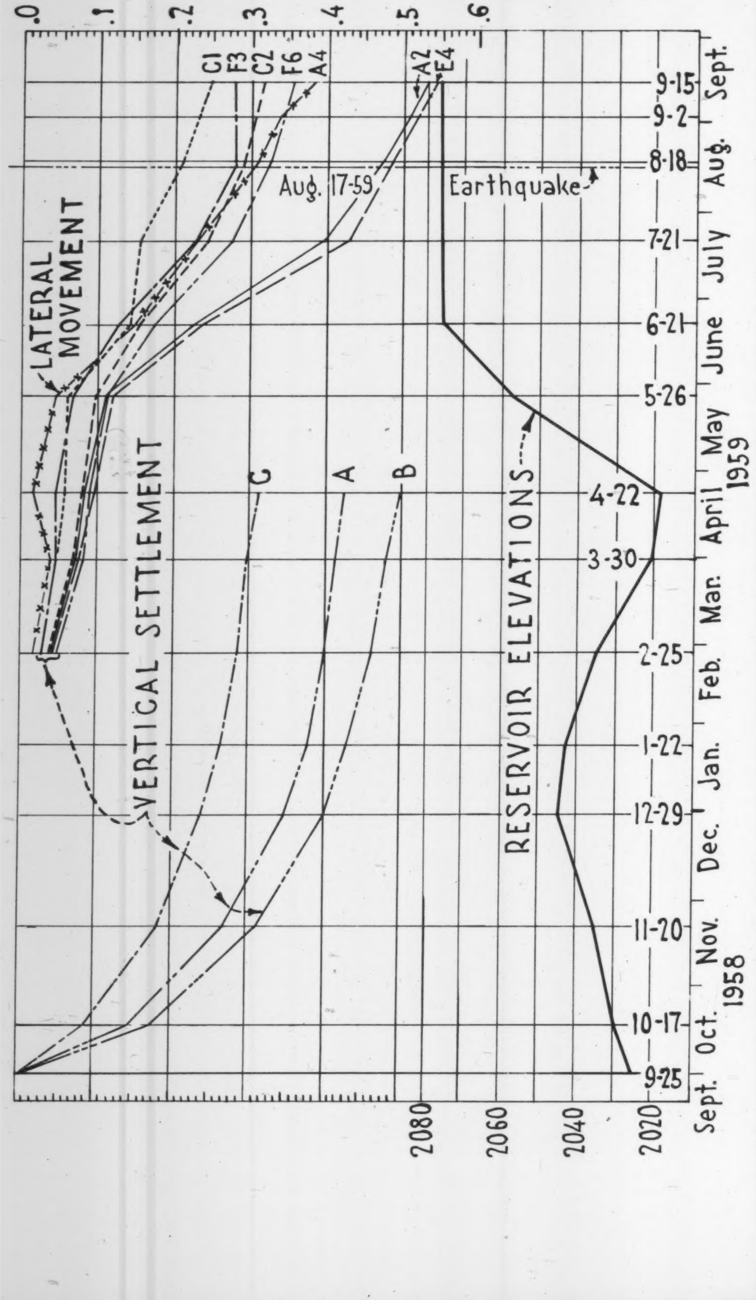

FIG. 18.—SETTLEMENTS

TABLE 4.—DAM HORIZONTAL MOVEMENT SINCE PREVIOUS MONTH

Mon No	Oct 17 1958	Nov 20 1958	Dec 29 1958	Jan 22 1959	Feb 25 1959	Mar 30 1959	Apr 22 1959	May 26 1959	June 21 1959	July 21 1959	Aug 18 1959	Sept 2 1959	Sept 15 1959	Total
HW	2029.0	2034.9	2044.6	2043.7	2034.7	2020.3	2018.2	2057.0	2076.1	2076.2	2076.5	2076.5	2076.3	26,664.0
A	.14	.13	.09	-.03	-.005	.005	-.05							.28
B	.21	.05	.09	.02	.04	.02	-.07							.36
C	.13	.04	.07	.025	.00	.005	-.06							.21
E-2-X												.00		.00
E-2-Y					.02	-.02	-.02	.03	.00	.04	-.01	.01	.00	.05
D-2-X												.12		.12
D-2-Y					.04	-.01	-.01	.05	.04	.06	.05	.00	.01	.23
C-2-X												.06		.06
C-2-Y					.00	.00	.00	.03	.04	.12	.04	.01	.00	.24
B-2-X												.17		.17
B-2-Y					.03	.03	-.04	.02	.13	.12	.04	.01	.01	.35
A-4-X												.06		.06
A-4-Y					.02	.02	-.02	.03	.11	.08	.08	.03	.05	.40
A-3-X												.03		.03
A-3-Y					.00	.03	-.04	.02	.10	.09	.10	.00	.01	.31
A-2-X												-.06		-.06
A-2-Y					.02	.01	-.02	.02	.06	.15	.03	.02	.05	.34
A-1-X												-.09		-.09
A-1-Y					.00	-.02	+.02	-.02	.11	.06	.04	.05	.02	.26
B-1-X												-.11		-.11
B-1-Y					.00	.01	-.05	.04	.04	.09	.08	.00	.00	.21
C-1-X												-.17		-.17
C-1-Y					.00	.01	-.01	.00	.06	.09	.00	.00	.00	.15
D-1-X												-.29		-.29
D-1-Y					.00	.03	-.03	.00	.04	.04	.01	.00	.03	.12
E-1-X												-.15		-.15
E-1-Y					.00	.00	.00	.00	.04	-.04	-.02	.03	.01	.02

TABLE 5.—DAM SETTLEMENT SINCE PREVIOUS MONTH

Mon No	Sept 25 1958	Oct 17 1958	Nov 20 1958	Dec 29 1958	Jan 22 1959	Feb 25 1959	Mar 30 1959	Apr 22 1959	May 26 1959	June 21 1959	July 21 1959	Aug 18 1959	Sept 15 1959	Total
HW	2023.9	2029.0	2034.9	2044.6	2043.7	2034.7	2020.3	2018.2	2057.0	2076.1	2076.2	2076.5	2076.3	26,611.4
A	0	.148	.123	.080	.028	.022	.013	.009						.423
B	0	.173	.140	.087	.033	.028	.019	.017						.497
C	0	.092	.089	.060	.023	.021	.015	.016						.316
E-2						.015	.014	+.001	.006	.045	.057	.023	.021	.180
F-8						.020	.019	.000	.009	.025	.050	.022	.021	.166
D-2						.033	.025	.007	.014	.038	.069	.039	.028	.253
F-7						.039	.029	.006	.020	.041	.079	.043	.032	.289
C-2						.039	.030	.010	.019	.053	.091	.043	.033	.318
F-6						.043	.034	.013	.021	.060	.105	.047	.033	.356
B-2						.040	.031	.012	.023	.068	.110	.061	.042	.387
F-5						.045	.034	.013	.027	.078	.123	.067	.048	.435
A-4						.041	.033	.015	.027	.090	.138	.068	.051	.463
A-3						.037	.032	.014	.032	.107	.156	.084	.058	.520
F-4						.042	.034	.015	.033	.113	.166	.084	.059	.546
A-2						.039	.030	.014	.029	.117	.169	.079	.058	.535
A-1						.038	.025	.008	.026	.112	.168	.081	.056	.509
B-1						.033	.020	.007	.019	.101	.146	.064	.045	.435
C-1						.028	.016	.007	.015	.075	.113	.055	.035	.344
F-3						.029	.017	.003	.014	.066	.102	.048	.030	.309
D-1						.026	.013	.006	.008	.040	.072	.027	.019	.211
F-2						.025	.013	.003	.008	.028	.062	.021	.013	.173
E-1						.014	.004	+.001	.006	.005	.030	.010	.005	.073
F-1						.013	.001	+.003	.007	.000	.021	.005	.001	.045

toward the center as the dam arch flattens. The comparative readings between some monuments seems to be questionable. Some check readings taken during the compilation of this data tends to bear out the magnitude of these readings based on the rate of change over a short period. An additional measurement system for this type of records has now been set up, with the hope of determining whether this inconsistency does exist or if the initial positions are in error axially.

Shortly before midnight on August 17, 1959, a severe earthquake occurred in the Hebgen dam area in Montana. The epi-center of this quake was reported to have been at a point approximately 300 miles from Brownlee dam. Intensity at the epi-center was variously estimated at 7.2 to 7.8 on the Richter scale. On August 18 readings were taken on all monuments and settlement rods along the crest of Brownlee Dam. Those readings did not reveal any effect of the earthquake on the rate of movement.

VILGOT LANNER.[9]—Mr. Mundal's paper gives a comprehensive and valuable engineering picture of the 400 ft high Brownlee sloping core rockfill dam. This paper was of special interest to the writer since his organization is presently engaged on the Trångslet Dam of similar type, height and yardage. Brownlee and Trångslet Dams are all of sloping core rockfill type, of approximately 400 ft height and each containing approximately 10,000,000 cu yd of material.

At the turn of the year 1959-60 the Trångslet dam had reached 75% of the total height; and storage was 250 ft. The entire project will be completed at the end of 1960. It is the intention of the writer's organization to make a detailed presentation of the engineering and early performance of the dam in 1961. Hitherto, only very brief international mention of the Trångslet Project has been made, viz. in the Engineering News-Record, Sept. 11, 1958, pages 60-66, (special issue on the occasion of the Sixth International Congress on large dams—ICOLD—in New York). However, the dam was demonstrated to the participants in the 26th Executive Meeting (ICOLD) on June 11, 1959. The entire project is planned, designed and constructed by the private company Stora Kopparbergs Bergslags AB., Falun, Sweden, under the management of Vilgot Lanner with Thorild Persson as chief designer, and Albert Andersson as chief resident engineer. During the visit to Trångslet printed information on the project was handed out to the participants. The writer presents herewith information concerning the dam in order to put technical data on these high rockfill dams together in the technical literature as a part of this very worthwhile Symposium. Rather than compare features of the dams, the writer will present data on Trångslet Dam and leave comparisons up to the reader.

GENERAL

The local conditions are such that the dam at Trangslet could be constructed either as a concrete gravity dam, or as an earth or rock fill-type dam. The fill-type of dam was decided on, in view of greater permanence. Since material was readily available near the site, a rockfill dam with impervious core was found to be technically and economically motivated. An inclined impervious core was given preference to a central core for the following reasons:

1. With an inclined impervious core it is possible to carry out the supporting rockfilling in advance, and this involves that initial settlements can develop without the impervious core having reached a point where it would be injuriously affected by such settlements.
2. Construction of a dam with a sloping core provides more regular employment for labour and machines than a dam with a central core in a climate where there is a limited season during which core material may be placed.
3. With an inclined core, the downstream rockfill may be sluiced with water without risk of damage to the filters.

At Trangslet the rock consists of syenite and porphyry with isolated sections of diabase. The surface layer of foundation rock had been affected by atmospheric decomposition, and extensive clearing operations were necessary, especially in the area covered by the impervious core.

Conditions at the site are such that the cost per cubic yard for rockfill and moraine will be approximately the same. This, together with the fact that the shear strength of the core material is high, has contributed to the decision that the impervious core should be relatively thick. A general view of the dam is shown in Fig. 19 and a cross section in Fig. 20.

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[9] Vice Pres. and Mgr., Power Dept., Stora Kopparbergs Bergslags Aktiebolag, Falun, Sweden.

In order to allow for future settlements, the impervious core is horizontally bow-formed with a maximum chord of 50 ft, and will, when completed, rise above the crest at the center of the dam by 5 ft over the horizontal plane.

Other principal data:

Height	120 m.	(400 ft)
Crest length	850 "	(2800 ft)
Width of base	550 "	(1800 ft)
Crest width	10 "	(33 ft)
Incline of upstream slope	1:2.5	
Incline of downstream slope	1:1.25	

Volumes

Rockfill, downstream	(ungraded rock)	3.25 mill. cu m (4.13 mill. cu yd)
Coarse filter, downstream	(moraine stone)	0.50 mill. cu m (0.64 mill. cu yd)
Medium filter, downstream	(graded crushed rock)	0.25 mill. cu m (0.32 mill. cu yd)
Fine filter, downstream	(graded gravel)	0.25 mill. cu m (0.32 mill. cu yd)
Impervious core	(graded moraine)	1.50 mill. cu m (1.91 mill. cu yd)
Filter, upstream	(ungraded gravel)	0.25 mill. cu m (0.32 mill. cu yd)
Supporting rockfill, upstream	(ungraded rock)	1.20 mill. cu m (1.52 mill. cu yd)
Total		7.2 mill. cu m (9.2 mill. cu yd)

Construction.—Work on the base rock.—All loose and decomposed rock must be removed from the area where the impervious core and the filters will contact the base rock. The surface of the rock is then cleared and concrete is added in order to obtain a continuous contact surface. To prevent ground water damaging the impervious core at the contact surface, the ground rock is sprayed with concrete, thus sealing all cracks.

Below the impervious core and in a position indicated in Fig. 20 a grout curtain has been made in the rock. The depth of these cement injections is determined by drilling tests and pressure measurements in drilled holes at various depths. Generally, injections have been made to a depth of about 65 ft (20 m).

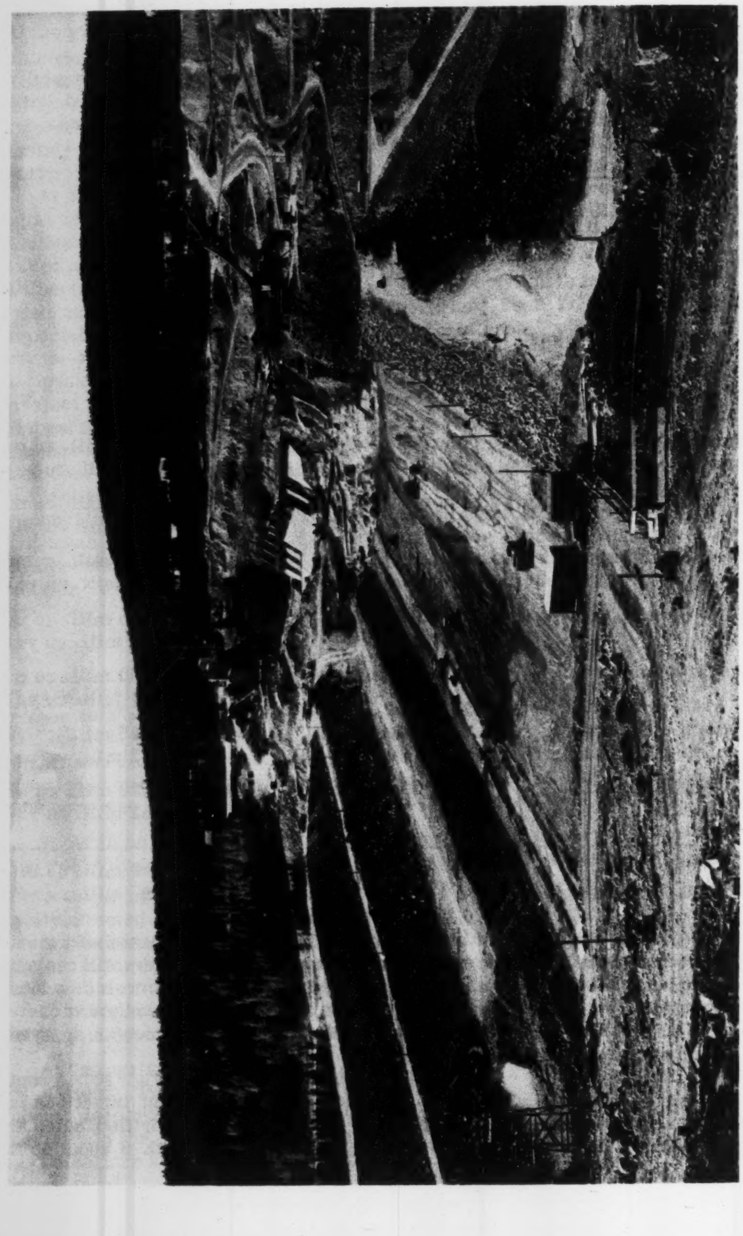


FIG. 19.—GENERAL VIEW - CONSTRUCTION OF TRANGSLET DAM

Some 65 ft (20 m) downstream from the injection curtain and at a depth of about 65 ft (20 m) an inspection gallery runs under the entire core, acting as a drain for the rock under the core. In the future, any necessary complementary injections under the dam can be carried out from this tunnel. The inspection gallery is drained into the discharge tunnel.

Fill Materials.—The rockfill consists chiefly of porphyry from the quarry upstream from the dam. Rock from the various underground operations—alternatively porphyry and diabase—is also put in the dam. In general, tunnel rock proved less suitable as rockfill than the other types of rock, by reason of the relatively high proportion of small-sized material.

Rock size:	<500	<200	<100	50 mm
Quarry rock	70	40	22	10 %
Tunnel rock	99	82	55	23 %

For this reason tunnel rock is normally used in the filters after crushing and grading.

The filter between the impervious core and the rockfill comprises three layers, viz. coarse, medium, and fine. The rock in the coarse filter consists of stones out of the moraine material. This rock normally varies in size from 100 to 200 mm and is very suitable for the coarse filter. Graded crushed rock from underground operation, sized between 15 and 80 mm, is used for medium filter. The fine filter is composed of river gravel to a maximum size of 50 mm. The grading of the material used in the filters is described in Fig. 21.

Material for the impervious core is taken from a moraine deposit close to the dam site. To render the moraine suitable for use as core material it is necessary to reduce the content of stone. This is done by rejecting all pieces over 100 mm. An average of 30% is sorted out and then used in the coarse filter, as described above. In this way a suitable core material is obtained with a clay content of approx. 3% ($<2\mu$). Further details of grading appear in Fig. 21. The core material has its optimum moisture content at 7 to 8%, calculated on the basis of material less than 16 mm. The moraine material has in general a permeability considerably lower than the value established as an upper limit, i.e. 0.10 cm/h. The shear strength of the material has been determined and the value of ϕ is 45°. Laboratory tests have also shown that this value will increase when the reservoir is filled. This is equally true whether the material is packed at Proctor optimum or by the wetpacking method. Laboratory tests have also been carried out to determine the effect which the variation of storage would have on the stability of the core material. These tests disclose that the variations have no appreciable effect on the degree of settlement.

Placing of Fill Materials.—The part of the rockfill, which transmits the load from the impervious core to the base rock, is sluiced so as to obtain a body of sound supporting material. Sluicing is carried out with high-pressure nozzles at a pressure of 20 atmos. (300 psi), the capacity of the pumps being 6000 l/min (1320 gal per min). To every cubic yard of rockfill 4 cu yd of water are used. Rockfill downstream from the supporting section is not sluiced. This part is built up during winter. The rockfill is dumped from fairly great heights,

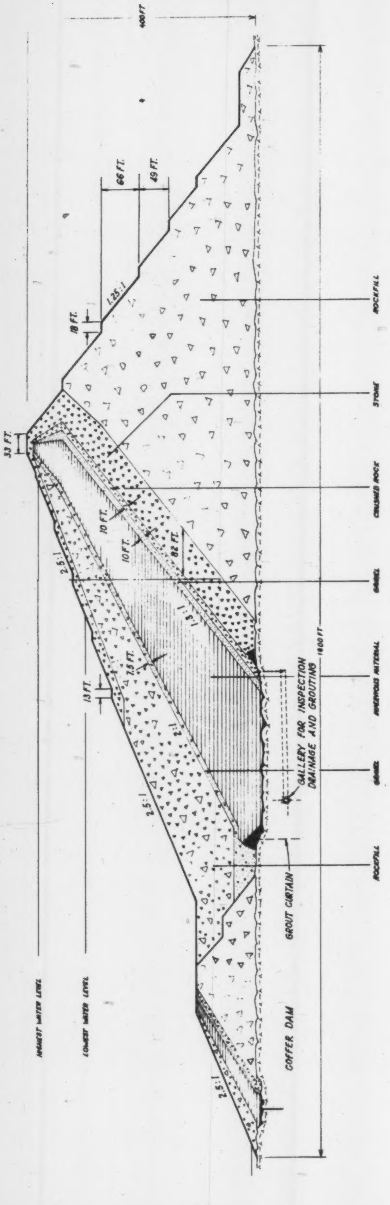
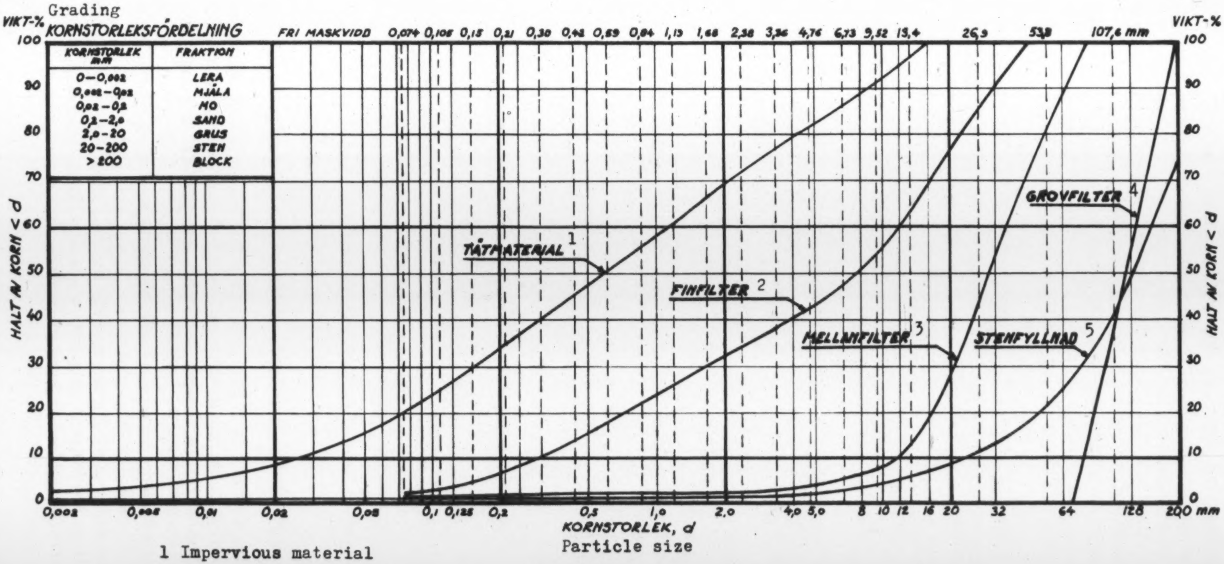

400 FT
66 FT.
49 FT.
18 FT.
1.25:1
33 FT.
10 FT.
10 FT.
82 FT.
1.8:1
2.5:1
7.5 FT.
13 FT.
2:1
2.5:1
1800 FT
GALLERY FOR INSPECTION DRAINAGE AND GROUTING
GROUT CURTAIN
HIGHEST WATER LEVEL
LOWEST WATER LEVEL
COFFER DAM
2.5:1
ROCKFILL
STONE
CRUSHED ROCK
GRAVEL
IMPERVIOUS MATERIAL
GRAVEL
ROCKFILL

FIG. 20.—CROSS SECTION TRANGSLET DAM



1 Impervious material

2 Fine filter material

3 Medium filter material

4 Coarse filter material

5 Rock-fill material

FIG. 21.—GRADING CURVES - TRANGSLET DAM

up to 20 m (65 ft) as experience indicates that settlement will be less when such heights are used. This applies particularly to rockfill that is dumped in winter.

The coarse filter is sluiced and then packed by tractors in a layer of 0.5 to 1 m thickness. The medium and fine filters are sluiced and packed in layers of 0.5 m thickness. The vibrator used in compaction is shown in Fig. 22.

The moraine material is compacted in 0.3 m loose layers (11.8 in.) by 40-ton rubber-tired rollers. Six rollings give over 95% effective compaction which is the minimum aimed at. During rolling, the moisture content is kept, if possible, around 7.5 to 8%, or somewhat over the Proctor optimum. The material is dampened when necessary. Experience shows that it is difficult to drive trucks and rubber-tired rollers over the material if the moisture content goes up to 10%. At a moisture content of 10.5% such operations are not practicable.

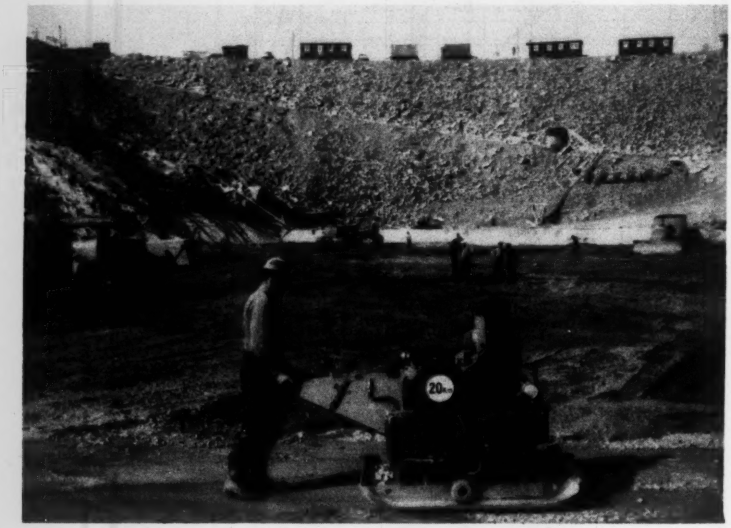


FIG. 22.—VIBRATING THE FILTERS, TRANGSLET DAM

The photograph of Fig. 23 shows a summer stage of core, filter and rockfill construction.

Control.—A continuous check is kept on selection and treatment of material. Special care is exercised over the quality and treatment of material for the impervious core. In the course of compaction, tests are made of the comparative degrees of compaction. A sample of such a test report is shown in Fig. 24. A modern aid to the continuous and rapid checking of packing is the isotope test, which gives quick and reliable results.

Observations.—The dam, during construction as well as for a long period after completion, will be the subject of various observations, including measurements of settlement, leakage and pore pressure.

Settlement.—As the successive stages of the dam are completed, settlement will be observed from a number of points at different levels. Measurements carried out hitherto have been mainly concerned with material placed in winter and thus not sluiced. There is evidence that settlement decreases rapidly as time goes on.

Leakage.—Leakage along the entire dam is expected to amount to 8 l per sec (1.8 gal per sec.) with a seepage rate of 0.10 cm per hr (0.04 in. per hr). Permeability has been found to be better than anticipated and the actual leakage might therefore be less than the value given. Leakage is measured at three different points:

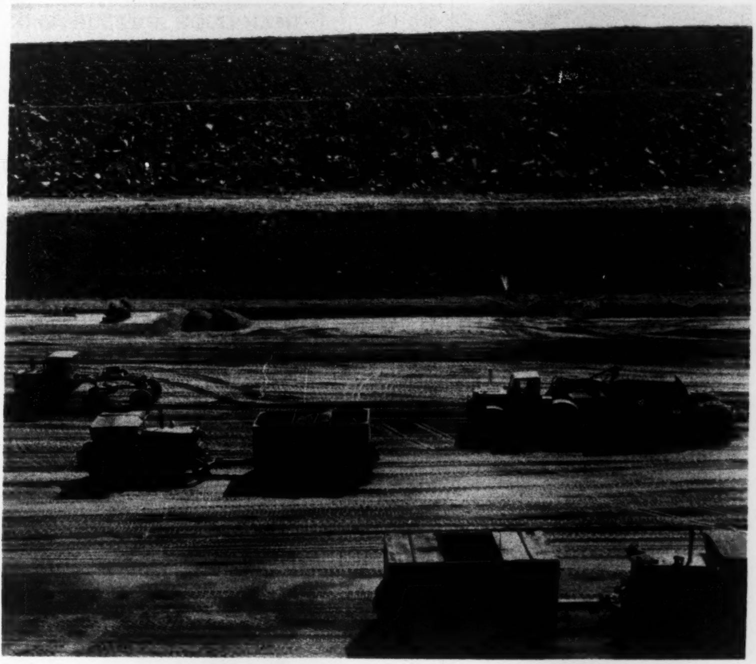
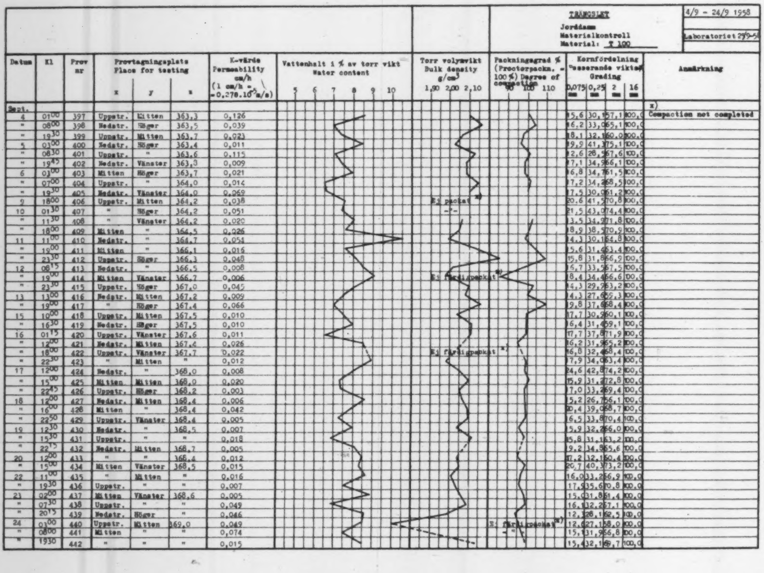


FIG. 23.—WORK ON THE CORE IN EARLY SUMMER 1959 - TRANGSLET DAM

1. A spillway has been arranged at the foot of the dam, in which all water is collected and measured. Correction for rainwater snow will be necessary.
2. Under the lowest section of the dam there is a gallery in which leakage water from the contact zone between the impervious core and the ground rock is collected and measured. This arrangement is already in use. At the present storage level upstream, which is 35 m (115 ft.), the leakage does not exceed 0.2 lit./sec. (0.044 gal.).

3. The rock under the core drains into an inspection gallery running under the entire length of the core. Measurements of leakage water are taken in vertical drainage wells located every 5 to 10 m (16.4 - 32.8 ft.) from each other.

Pore Pressure Measurements.—Instruments are being introduced into the interior of the core, to enable pore pressures to be measured at different points in the body of the dam. Similarly, pressure measurements are made in fissure areas in the base rock under the dam. The pore pressure measurements are checked from the inspection gallery. These measurements will be of value in determining the significance of the speed of storage. The normal regulating height will in future be 25 m (82 ft.) and the maximum water level variation is expected to be 1 m. per day.



TRÅNGSLET
Jorddamm
Materialkontroll
Material: T 100

4/9 - 24/9 1958
Laboratoriet 27/9-5

Datum	Kl	Prov nr	Provtagningsplats Place for testing x	y	z	K-värde Permeability cm/h (1 cm/h = = 0,278.10^{-5} m/s)	Vattenhalt i % av torr vikt Water content 5 6 7 8 9 10	Torr volymvikt Bulk density g/cm^3 1,90 2,00 2,10	Packningsgrad % (Proctorpackn. = 100 %) Degree of compaction 90 100 110	Kornfördelning Passerande vikt% Grading 0,075 mm	0,25 mm	2 mm	16 mm	Anmärkning
Sept.														x)
4	01^{00}	397	Uppstr.	Mitten	363,3	0,126				15,6	30,1	57,1	100,0	Compaction not completed
"	08^{00}	398	Nedstr.	Höger	363,5	0,039				16,2	33,0	65,1	100,0	
"	19^{30}	399	Uppstr.	Mitten	363,7	0,023				18,1	32,1	60,0	100,0	
5	03^{00}	400	Nedstr.	Höger	363,4	0,011				19,9	41,3	75,1	100,0	
"	08^{30}	401	Uppstr.	"	363,6	0,115				12,6	28,5	67,6	100,0	
"	19^{45}	402	Nedstr.	Vänster	363,3	0,009				17,1	34,9	66,1	100,0	
6	03^{00}	403	Mitten	Höger	363,7	0,021				16,8	34,7	61,5	100,0	
"	07^{00}	404	Uppstr.	"	364,0	0,014				17,2	34,2	68,5	100,0	
"	19^{30}	405	Nedstr.	Vänster	364,0	0,069				17,5	30,0	61,2	100,0	
9	18^{00}	406	Uppstr.	Mitten	364,2	0,038				20,6	41,5	70,8	100,0	
10	01^{30}	407	"	Höger	364,2	0,051				21,5	43,0	74,4	100,0	
"	11^{30}	408	"	Vänster	364,2	0,020				13,5	34,9	71,8	100,0	
"	18^{00}	409	Mitten	"	364,5	0,026				18,9	38,5	70,9	100,0	
11	11^{00}	410	Nedstr.	"	364,7	0,054				14,3	30,1	64,8	100,0	
"	19^{00}	411	Mitten	"	366,1	0,016				15,6	31,4	63,4	100,0	
"	23^{30}	412	Uppstr.	Höger	366,3	0,048				15,8	31,8	66,9	100,0	
12	08^{15}	413	Nedstr.	"	366,5	0,008				16,7	33,5	67,5	100,0	
"	19^{00}	414	Mitten	Vänster	366,7	0,006				18,4	34,4	66,6	100,0	
"	23^{30}	415	Uppstr.	Höger	367,0	0,045				14,3	29,9	63,2	100,0	
13	13^{00}	416	Nedstr.	Mitten	367,2	0,009				14,3	27,6	59,3	100,0	
"	19^{00}	417	"	Höger	367,4	0,066				19,8	37,6	68,4	100,0	
15	10^{00}	418	Uppstr.	Mitten	367,5	0,010				17,7	30,9	60,1	100,0	
"	16^{30}	419	Nedstr.	Höger	367,5	0,010				16,4	31,4	59,1	100,0	
16	01^{15}	420	Uppstr.	Vänster	367,6	0,011				17,7	37,8	71,9	100,0	
"	12^{00}	421	Nedstr.	Mitten	367,4	0,026				16,2	31,9	65,2	100,0	
"	18^{00}	422	Uppstr.	Vänster	367,7	0,022				16,8	32,4	68,4	100,0	
"	22^{30}	423	"	Mitten	"	0,012				17,9	34,0	63,4	100,0	
17	12^{00}	424	Nedstr.	"	368,0	0,008				24,6	42,8	74,2	100,0	
"	15^{00}	425	Mitten	Mitten	368,0	0,020				15,9	31,2	72,8	100,0	
"	22^{45}	426	Uppstr.	Höger	368,2	0,003				17,0	33,2	69,4	100,0	
18	12^{00}	427	Nedstr.	Mitten	368,4	0,006				15,2	26,7	56,1	100,0	
"	16^{00}	428	Mitten	"	368,4	0,042				20,4	39,0	68,7	100,0	
"	22^{50}	429	Uppstr.	Vänster	368,4	0,005				16,5	33,8	70,4	100,0	
19	12^{30}	430	Nedstr.	"	368,5	0,007				15,9	32,2	66,0	100,0	
"	15^{30}	431	Uppstr.	"	"	0,018				15,8	31,1	63,2	100,0	
"	22^{15}	432	Nedstr.	Mitten	368,7	0,005				19,2	34,9	65,6	100,0	
20	12^{00}	433	"	"	368,4	0,012				17,2	32,1	60,4	100,0	
"	15^{00}	434	Mitten	Vänster	368,5	0,015				20,7	40,3	73,2	100,0	
22	11^{00}	435	"	Mitten	"	0,016				16,0	33,2	66,9	100,0	
"	19^{30}	436	Uppstr.	"	"	0,007				17,9	35,6	70,8	100,0	
23	02^{00}	437	Mitten	Vänster	368,6	0,005				15,0	31,8	61,4	100,0	
"	07^{30}	438	Uppstr.	"	"	0,049				16,1	32,2	67,1	100,0	
"	20^{15}	439	Nedstr.	Höger	"	0,046				12,3	28,1	62,5	100,0	
24	03^{00}	440	Uppstr.	Mitten	369,0	0,049				12,6	27,1	58,0	100,0	
"	08^{00}	441	Mitten	"	"	0,074				15,1	31,9	66,8	100,0	
"	1930	442	"	"	"	0,015				15,4	32,1	69,7	100,0	

FIG. 24.—CORE MATERIAL TEST REPORT, TRANGSLET DAM

RATES OF WORK

In order to follow the schedule of work, the following average rates will have to be maintained:

		Winter (6 months)	Summer (6 months)
Rockfill	(daily)	6.000 cu m (7.600 cu yd)	3.000 cu m (3.800 cu yd)
Filters (all)	"	-	3.000 cu m (3.800 cu yd)
Core material	"	-	5.000 cu m (6.400 cu yd)

CONSTRUCTION EQUIPMENT

The most important equipment used is:

Excavators	3	4.5 cu m (6.0 cu yd) Marion
	2	1.5 " " (2.0 " ") Ruston-Bucyrus 38
	1	1.0 " " (1.3 " ") " " 30
	1	0.5 " " (0.6 " ") " " 19
	4	0.9 " " (1.2 " ") Landsverk
Total	11	

Tractors	6	Caterpillar D8
	4	" D7
	2	" D6
	1	" D4
Total	13	

	11	25-ton Dart
Trucks	14	20-ton Faun
	13	15-ton Faun
	4	7-ton Koering
	20	5/6-ton of various makes
Total	62	

Dumpers	3	20-ton Caterpillar DV21
	1	20-ton Euclid
Total	4	

Rubber-tired rollers	3	40-ton
Vibration machines	5	Bohn und Kähler

Labor.—During the summer half-year, when work is carried on in three shifts, 3 x 110 = 330 men work on the dam. More than 30 cu m per man and day is then brought to the dam, that is, about 4 cu m per man per hr.

Costs.—On the basis of performance hitherto, costs for the various materials are expected to amount to the values shown in the table below.

Material	Length of transport miles	Volume mill cu yd	Capacity cu yd per hr	Unit price exclus. costs for administration		
				£ /cu yd	$ / cu yd	Sw kr/ m^3
Rockfill from quarry (blasting loading transport sluicing)	1	6.0	400	6 sh	0.85	6
Coarse filter moraine stone (transport sluicing compaction)	1	0.7	130	5 sh	0.70	5
Medium filter crushed rock from the tunnels (transport crushing screening sluicing compaction)	4	0.35	70	12 sh	1.70	12
Fine filter graded gravel (transport screening sluicing compaction	7	0.35	70	11 sh	1.60	11
Impervious material graded moraine (transport screening compaction)	2	2.0	330	8 sh	1.15	8
Filter	7	0.35	70	8 sh	1.15	8

Mean-price without costs for administration 7 sh/ cu yd = 0.95 $ / cu yd = 6.80 kr/ m

No foundation work is included in these costs, nor do the calculations include costs for central administration.

J. B. COOKE,[10] F. ASCE.--The writer would like to emphasize: (a) that rockfill dams can be safely supported on consolidated stream bed material; and (b) that rock not suitable for dumped rock may be utilized by compacting it in layers. Both of these features have been adopted at Brownlee Dam and at Cougar Dam. Both features are being incorporated in more rockfill dams as the technology advances rapidly and rockfill dams are more frequently adopted. Two dams adopting these economical features are Goschenen Dam in Switzerland, and Messaure Dam in Sweden; both of these dams are now (1960) in an advanced stage of construction. The writer will briefly present data on these two interesting dams using recent information provided by Albert Luchinger of Locher & Cie, Switzerland, and by Tore Nilsson of the Swedish State Power Board. A third major dam is Furnas Dam in Brazil.

This discussion also comments on Brownlee settlement and on the definition of rockfill dams.

Goschenen (Goschenenalp) Dam—Switzerland.--Goschenen Dam is a thin central core rockfill dam of 155 m (510 ft) height and 540 m (1770 ft) crest length. The dam contains 9,000,000 cu m (11,700,000 cu yd) of material and creates a 60,000 acre-ft reservoir. Fig. 25 shows the cross section of the dam. References provide information on composition of the core,[11] on shear strength and test fills of the shell rockfill material,[12,13] and on the construction.[14,15] Construction started in 1955, but due to a 5-month summer construction season, the dam is only now (1960) nearing completion; the whole watershed in this portion of the Swiss Alps is subject to avalances. The rockfill design was estimated to cost one-half as much as a concrete gravity dam.[14] The downstream slope with 1.45: 1 slopes and benches averages 1.65: 1 for the top 70.5 m (232 ft) height of the dam. This is steeper than has previously been used for this type and height of dam. McKeer has presented[15] many illustrations and grading curves.

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[10] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

[11] "Goeschenalp Rockfill Dam Project, Switzerland," by W. Eggenberger, Proceeding, 3d Internatl. Conf. on Soil Mechanics and Foundations Engrg. Vol. 3, 1953, p. 296.

[12] "Shear Strength of the Materials for the Supporting Shell of the Goeschenenalp Dam, Switzerland," by J. Zeller and R. Wulliman, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957, p. 399.

[13] "Test Fill With Coarse Shell Material at Goschenenalp Dam, Switzerland," by J. Zeller and H. Zeindler, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957, p. 405.

[14] "Die Baulichen Analgen Des Kraftwerke Goshenen," ("The Civil Construction Features of the Goshenen Power Project"), by Rudolf Pfister, Die Wirtschaft, July, 1959, p. 35.

[15] "Construction Methods for Goeschenen Dam," by Harold J. McKeever, World Construction, September, 1960, pp. 16-33.

Foundation.—The foundation for the core was carried to good granite. The depth of excavation was as great as 230 ft, and amounted to 1,300,000 cu yd. Because the granite was particularly jointed, area grouting was resorted to in addition to the main grout curtain. The grout consisted of a mixture of Portland cement and Opalinus clay. The rockfill rests on granite or on well compacted sand-gravel deposits. The ballast layer, 8 in Fig. 25, rests on unreliable layers of clay, sand, peat, and organic matter. The settlement of the ballast zone amounted to 40 in. in 2 yr. The vertical drains, 30-in. diameter, gravel-filled holes, and the drainage bed under the possibly impervious zone 8 are to insure drainage during reservoir drawdown.

Impervious Zone.—The impervious zone is thin because it was necessary to manufacture the material from local sand and gravel and imported clay. The clay is Opalinus clay from the Jura mountains and is used in an amount of 875 lb per cu yd. The material is mixed in a specially built plant and its quality, consistency, and plasticity are rigidly controlled.

Rockfill.—The rockfill material is from Talus deposits. It is not processed and contains a high percentage of fines. Rocks larger than 1 cu m are not used in the fill. Grading curves were made of the natural deposits using 20 cu m to 30 cu m (26 cu yd to 39 cu yd) of material for each sample and up to the 1 cu m blocks. The high percentage of fines is illustrated by the following data; 30% smaller than 10 mm size (0.4 in.) and 44% smaller than 100 mm (4.0 in.) size.[4] Another measure of the high percentage of fines is the low percentage of voids in the fill, 17% to 18%, and the dry density of 2.25 tons per cu m (141 lb per cu ft) for the rock of 2.72 specific gravity. The 17% to 18% voids for the fill containing a high percentage of fines compares with 27% to 30% for cleaner rockfill placed in layers and 30% to 32% for larger rockfill dumped in high lifts. The rockfill at Goschenen Dam is dumped wet in layers 2 m thick and not exceeding 3 m-thick, with no sluicing and with no compaction other than by hauling and spreading equipment. The method of placing rock was based on data from nine test fills containing a total of 30,000 cu m of material and located within the dam.[13]

Messaure Dam.—Messaure Dam is a thin central core rockfill dam of 101 m (330 ft) height. The dam is the largest rockfill dam in Sweden, containing 10,500,000 cu m (13,800,000 cu yd); the highest is the 400-ft Trangslet Dam. Messaure Dam is located on the Stora Lule River 7 miles north of the Arctic Circle. Due to a 5-month summer construction season, the project is taking 5 yr to complete; construction is to be finished in 1962. A description of Messaure Dam and three smaller Swedish rockfill dams, Ligga, Harspranget, and Jarkvissle, is available.[14] Fig. 26 shows the main river cross section and the cross section for the two abutments. Messaure Dam is of interest because of its economical use of available unprocessed materials, use of foundation excavation silt-gravel deposits in the embankment, and use of required project excavations for the rockfill. The decision to carry the abutment sections to bedrock was influenced by the economics of using much of the excavated materials in the (B) and (C) zones of the dam.

Core Foundation.—The foundation for the moraine core is carried to sound rock; all protruding rock formations and precipitous faces are removed so as to prevent any irregular settlements and to provide a longitudinal slope of the dam not exceeding 1:1 with vertical ledges not exceeding 0.5 m high; no foundation slope is to exceed 1:1 slope; and cavities and crevices are thoroughly washed out and filled with concrete and fissured rock covered with concrete to present an even surface for beginning compaction of the core. The minimum grout curtain consists of two parallel lines spaced 2 m (6.6 ft) apart, with holes at 1 m apart in the downstream row and 2 m apart in the upstream row. The work on the downstream row is completed before the upstream row is started. Two stages; the first to 5 m (16.4 ft), and the second 5 m to 12 m (16.4 ft to 33.5 ft) are used. Water washing, water testing, and grouting are thoroughly carried out.

Zones.—The zones of Fig. 26 are as follows:

A. Moraine Core.—The material is obtained from moraine pits that meet specified clay content, grading and permeability limits. Rocks up to 10 cm (4 in.) size are permitted in rolled fill and up to 25 cm (10 in.) in fill placed by the "wet compaction method," but the sizes are subject to change should the amount of rocks warrant it. The "wet compaction method," developed by the Swedish State Power Board for use with moraines, is described by Mr. Nilsson.[14] It involves compaction of the moraine at a water content near the liquid limit of the material. This requires a material with a limited clay content that is, therefore, reasonably permeable. For Messaure, the moraine complies with the following: the clay content, that portion of the material with grain less than 0.006 mm, must not exceed 10% of material less than 5.6 mm; the fine material content, the portion of the material with grain less than 0.076 mm, must amount to at least 20% of material less than 5.6 mm; and the permeability k-value of the moraine must be between 0.1-0.005 cm per hr determined under a normal pressure of 2 kg per sq cm. The moraine as a rule is placed by the "wet compaction method," the compaction being done by a caterpillar size D-8 or heavier. Where rolled fill is used, it is compacted to at least 95% of the maximum degree of compaction obtained with the modified Proctor method.

B. Transition Zone.—The material consists of coarse silt-sand. Material less than 0.06 mm does not exceed 15% of material less than 5.6 mm. The K-value is specified to preferably be 25-50 times greater than the K-value for moraine. Filter requirements to the (a) zone are complied with. Compaction is to at least 96% of maximum compaction obtained with the modified Proctor method.

C. Gravel Fill.—The material is sand or gravel which after specified compaction shall have a friction angle of at least 35°. The material is placed in layers and compacted at optimum water content to 95% modified Proctor compaction.

D. Supporting Fill (Pervious).—The material consists of sand-gravel with sufficient rock that after specified compaction has an angle of friction of about 40° above and 35° below El. 141. Some of the material comes from abutment excavation. The material is placed in layers and compacted at optimum water content to 95% modified Proctor compaction. Where necessary, filter layers are provided between (D) and (B) zones. Rockfill may be used in the (D) zone.

E. Rockfill (Quarried Rock).—The rockfill material is obtained from a spillway channel, the underground powerhouse and the tail-race tunnel. The grading is specified to contain all sizes with a maximum size of 1 m and with the amount of fines not to exceed 20% smaller than 20 cm. The rockfill is

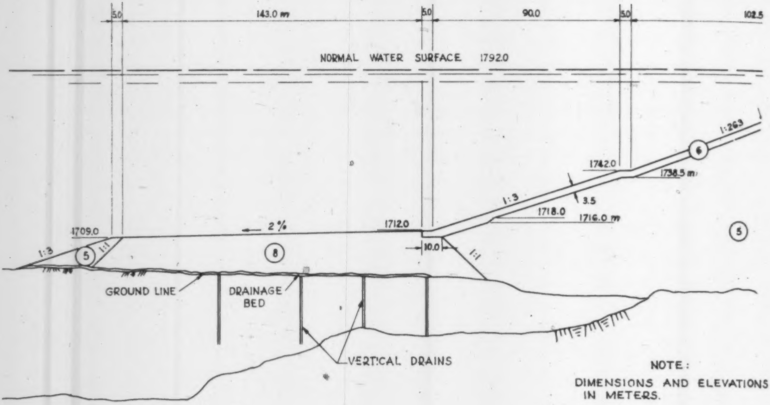

5.0
143.0 m
5.0
90.0
5.0
102.5
NORMAL WATER SURFACE 1792.0
1:263
⑥
1742.0
1738.5 m
1:3
3.5
1718.0
1716.0 m
1709.0
2 %
1712.0
⑤
1:3
⑤
1:1
⑧
10.0
1:1
GROUND LINE
DRAINAGE BED
VERTICAL DRAINS
NOTE:
DIMENSIONS AND ELEVATIONS IN METERS.

FIG. 25.—CROSS SECTION

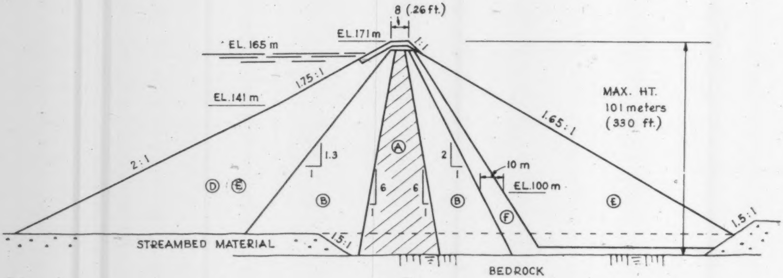
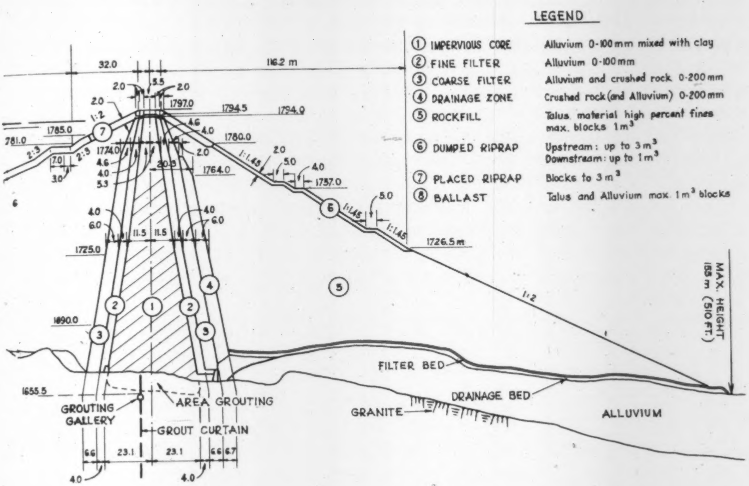
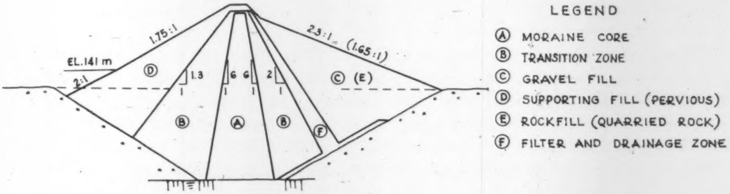

8 (.26 ft.)
EL.171 m
EL. 165 m
1:1
1.75 : 1
EL.141 m
MAX. HT.
101 meters
(330 ft.)
1.65 : 1
Ⓐ
1.3
1
2
1
2 : 1
10 m
EL.100 m
Ⓓ Ⓔ
Ⓑ
6
1
6
1
Ⓑ
Ⓔ
Ⓕ
1.5 : 1
STREAMBED MATERIAL
1.5 : 1
BEDROCK

FIG. 26.—CROSS SECTION


LEGEND
① IMPERVIOUS CORE Alluvium 0-100mm mixed with clay
② FINE FILTER Alluvium 0-100mm
③ COARSE FILTER Alluvium and crushed rock 0-200mm
④ DRAINAGE ZONE Crushed rock (and Alluvium) 0-200mm
⑤ ROCKFILL Talus material high percent fines max. blocks 1m³
⑥ DUMPED RIPRAP Upstream: up to 3m³ Downstream: up to 1m³
⑦ PLACED RIPRAP Blocks to 3m³
⑧ BALLAST Talus and Alluvium max. 1m³ blocks
32.0
116.2 m
5.5
2.0
1797.0
1794.5
1794.0
1785.0
781.0
1780.0
1774.0
1764.0
1757.0
1726.5 m
1725.0
1690.0
1655.5
1:2
2:3
1:1.45
FILTER BED
DRAINAGE BED
GRANITE
ALLUVIUM
GROUTING GALLERY
AREA GROUTING
GROUT CURTAIN
MAX. HEIGHT 155 m (510 FT.)

OF GOSCHENEN DAM


LEGEND
Ⓐ MORAINE CORE
Ⓑ TRANSITION ZONE
Ⓒ GRAVEL FILL
Ⓓ SUPPORTING FILL (PERVIOUS)
Ⓔ ROCKFILL (QUARRIED ROCK)
Ⓕ FILTER AND DRAINAGE ZONE
EL. 141 m
1.75:1
2.3:1 (1.65:1)
2:1
1.3
6
2

OF MESSAURE DAM

placed in 1 m to 1.5 m layers, compacted by hauling equipment and sluiced with water at a ratio of 2 volumes of water to one of rock. In a 4 m width adjacent to the (B) zone, the maximum rock size is 0.5 m, the maximum layer thickness is 1 m, and placing is not permitted if there is any possibility of freezing. During rock placing under freezing conditions in winter months, sluicing is not used and fines are further limited.

F. Filter and Drainage Zone.—The (F) zone between zones (B) and (E) consists of two zones, common gravel and quarried rock of 25 cm (10 in.) maximum size. Excessive fines are removed where necessary. For the (F) zone between zones (B) and (C) a single (F) zone of natural gravel may be used. Filter grading requirements must be complied with. The permeability K-value is specified to be, preferably, at least 25-50 times as great as the K-value for the (B) material, and that of the (F) zone rock, to be greater than the (F) zone gravel.

The thickness of layers and the number of passes of the various types of compaction equipment, for zones (B), (C), (D), and (F) are determined by compaction tests.

Brownlee Dam.--In his discussion Mr. Condit has presented valuable settlement measurements indicating that Brownlee Dam has performed very well. However, Mr. Condit noted that the crest monuments were installed several months after completion of the rockfill; the water level was already high when the first readings were taken; Mr. Mundal in his closure (to be presented subsequently) stated, "Maximum foundation and embankment settlement at Brownlee was 0.546 at Mounment F-4 for the recorded period; less than 0.2%." Though the foregoing points are all true, the reader might not get a true picture; the following tabulation is made to show the settlement of point A2 from Figs. 17 and 18 of Mr. Condit's discussion:

	Water Depth	Vertical Settlement
(a) About July, 1958 (Complete File)	?	?
(b) 9-25-58 (Begin A, B, and C bench reading)	225	0.46 (B)
(c) 2-25-59 (Begin crest measurements)	235	0.55 (A2)
(d) 9-15-59 (Recorded crest measurements)	275	
		1.01 plus

The water depths are above the stream bed El. 1800. The 1.05 ft does not include the first months of aging of the completed fill, the filling to 225 ft of the 275 ft full reservoir depth, and the early settlement in the top 40 ft of dam between the ABC bench at El. 2050 and the crest at nominal El. 2090. In the first recorded month for point B, it settled 0.18 ft with little raise in reservoir level. It is probable that in each of the several preceding months the settlement was greater. The data presented are valuable, but it is emphasized that the 0.546 (0.55) ft, for example, does not represent the settlement for the early aging of the completed fill and for the first filling.

It is agreed with Mr. Condit that it is not easy to establish permanent settlement points as soon as desired. However, the lack of complete information at Brownlee, and also at Mud Mountain Dam, suggests that temporary points should be established on such major dams in order that the complete record may be obtained. Even with the loss of a number of temporary points, a complete record can be developed.

Definition of Rockfill Dam.—In considering Goeschenen, Messaure, and Furnas dams to be rockfill dams, the definition must include rockfill compacted in layers. In the paper by John B. Snethlage, et al.,[16] the authors define a rockfill dam: "A rockfill dam is one of which at least half the materials in the maximum cross section is comprised of loose rock placed by dumping." It is possibly not clear whether "loose rock placed by dumping" includes rockfill that is dumped and compacted in layers. Another point in the foregoing definition on which the writer does not agree is the use of "at least half." At least half is, perhaps, too specific.

The writer considers a possible definition to be: "A rockfill dam is one that relies on rockfill, dumped in lifts or compacted in layers, as a major structural element."

TORALD MUNDAL,[17] F. ASCE.—The writer gratefully acknowledges the contributions of Messrs. Leonard, Lawton, Hsu, Ito, Condit, Lanner, and Cooke.

As indicated by Mr. Leonard, the combined cost of excavation and rock in place was unusually low, principally due to the fact that nearly all rock fill was furnished from required excavation. Related factors contributing to the economy of the operation were:

1. Little or no overburden had to be stripped.
2. Blasting was skillfully performed.
3. Excavated rock material required no processing, and there was no waste.
4. There were no long hauls.
5. Because operations were well scheduled, no stockpiling or rehandling of rock was necessary.

Mr. Lawton's question concerning ". . . specially drilled and grouted holes near the left abutment," was well taken. As described in the paper, foundation treatment included both area and curtain grouting in the core trench area. Area grouting was accomplished by means of shallow holes on ten-foot centers each way. Curtain grouting utilized a line of deeper holes. As drilling and grouting operations progressed, it became apparent that the treatment had to be supplemented in the left abutment area. Consequently, a number of additional holes, as much as 300 ft deep, were diamond drilled and grouted to stem the artesian flow. Thus, these holes were "special" in the sense that they were not originally planned; they were substantially deeper than the grout curtain holes; and they had no specific pattern or depth, inasmuch as requirements were dictated entirely by field conditions. Locations and depths were determined by experienced field engineers.

Mr. Lawton suggests additional discussion concerning rockfill sizes and the effects of diversion on the fine and coarse filter zones. With respect to sizes of rockfill, the exposed material at the time of diversion was within the range of 6 to 36 in. As was the case with the impervious core, both the fine and coarse filter zones were at an elevation about 10 ft lower than the downstream rockfill. Since the channel through the rockfill was restricted, the water pond-

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[16] "Rockfill Dams: Review and Statistics," by John B. Snethlage, F. W. Scheidenhelm and Arthur N. Vanderlip, Transactions, ASCE, Vol. 125, Part II, 1960.

[17] Vice Pres. and Chf. Engr., Internatl. Engrg. Co., Inc., San Francisco, Calif.

TABLE 6

	Unit	BROWNLEE	MIBORO	TRANGSLET
Height	ft	400	426	400
Crest Length	ft	1400	1400	2800
Core Thickness (average)	ft	30	93	85
Slopes				
Rockfill, U/S Face	--	1:3.0 (avg)	1:2.5	1:2.5
Rockfill, D/S Face	--	1:1.4 (avg)	1:1.75	1:1.5 (avg)
Impervious Core, U/S Face	--	1:1.5 and 1:1.8	1:1.5	1:2.0
Impervious Core, D/S Face	--	1:1.37	1:0.85	1:1.3
Volumes				
D/S Rockfill	cu yd	4,126,000	4,880,000	4,130,000
U/S Rockfill	cu yd	2,438,000	2,310,000	1,520,000
Filters	cu yd	468,000	1,100,000	1,600,000
Impervious Core	cu yd	449,000	2,140,000	1,910,000
Total Volume	cu yd	7,481,000	10,400,000	9,160,000

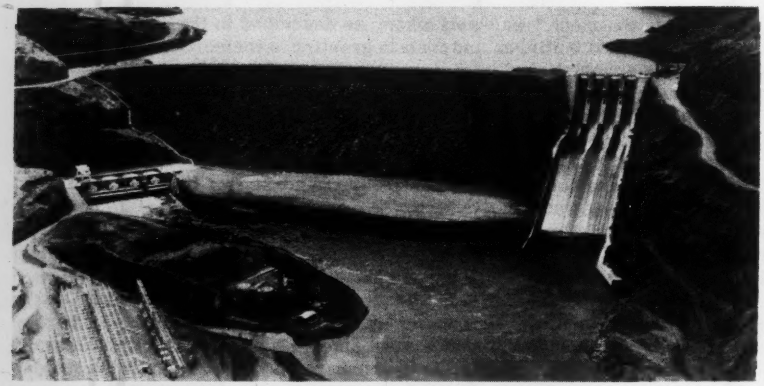

FIG. 27.—BROWNLEE HYDROELECTRIC PROJECT

ed over the core and filter area, and due to relatively low velocities, silt and cofferdam materials were deposited. After clean-up, it was found that effects of diversion flows were negligible, in that the filter and core zones had not been measurably disturbed.

Mr. Lawton requests justification for the statement, "unit costs for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago." An examination of contract unit prices for other rockfill projects constructed since 1945, will reveal that combined costs ranged from \$3 to \$5. Unit costs for excavation and embankment have, in effect, decreased in recent years because of greatly increased equipment capacities and efficiencies. For Brownlee Project, the factors enumerated at the beginning of this discussion effected economies sufficient to lower combined excavation and embankment operations costs to the extent indicated.

Mr. Condit's discussion of dam movement after completion is of great interest, in that the observations indicate that movement has been somewhat less than what might normally be expected. For example, it is generally felt that settlement of a rockfill embankment can, as a reasonable limit, approach one per cent of the height of the fill. Maximum foundation and embankment settlement at Brownlee was 0.546 ft at Monument F-4 for the recorded period; less than 0.2%. Settlement will undoubtedly continue for a number of years. However, inasmuch as the reservoir has been completely filled since June, 1959, most of the settlement has already occurred, and any future settlement will be at a much lower rate. It would appear, then, that (1) results of foundation investigations were correctly interpreted, and that foundation preparation and treatment were properly designed and constructed; and (2) the various embankment zones—core, filters, and rockfill—were well selected and well constructed.

Mr. Hsu's presentation of stability analysis details is a welcome addition to the discussion. This information, which was only briefly covered in the original paper, should be of great interest to designers of rockfill dams.

Descriptions of Miboro and Trangslet dams by Messrs. Ito and Lanner provide interesting technical data for inclusion with the discussion. Table 6 presents readily available comparative data for Brownlee, Miboro, and Trangslet. Note that the Brownlee impervious core is substantially thinner than those at Miboro or Trangslet. This may be attributed to the fact that physical characteristics and availability of Brownlee core material were such that design of a relatively thin core could be accomplished.

Significant savings on excavation and rockfill volumes were effected at Brownlee by designing the rockfill to be founded on as much as 100 ft of existing consolidated sand and gravel, rather than bedrock. As pointed out previously, settlement rates have been low, indicating that the foundation material is, for all practical purposes, as satisfactory as bedrock.

The writer is pleased to present information on Furnas Dam as suggested by Mr. Cooke.

A paper on the overall project consisting of a dam, spillway, power intake, penstocks, and powerhouse has been presented previously.[18] The dam, now under construction, will be a composite fill-type structure containing approximately 9,100,00 cu m (12,000,000 cu yd) of material. It will have a crest length of some 580 m (1,900 ft) and will have a maximum height of about 120 m (395 ft) above the river bottom.

The left abutment of the Furnas site is composed of a finely grained, highly jointed quartzite. The surface of suitable bedrock averages from 5 m to 10 m in depth below natural ground surface over most of the left abutment except at the spillway area where maximum depth is about 35 m (115 ft). The general strike is North 25° West, with the dip 15° to 20° to the Southwest, or downstream and away from the river channel. Bedding planes form joints consisting of very thin layers of clay or fine silts which have been metamorphosed into micaceous planes.

In the main river channel there is a deep, diagonal, fault-formed trough. This has been intruded by a diabase dike that has gouge and fractured rock on both sides.

The right abutment is composed of thick beds of thinly laminated quartzites separated by relatively deep bands of micaceous schists that are fractured and friable near the surface. There has been considerable decomposition and weathering of the schists and a talus slope has formed at this abutment.

Areas containing decomposed schist are present at both abutments. Joints in the quartzites usually are tight and little grouting will be required in most areas. Water testing indicates that little difficulty will be experienced with leakage.

As investigations progressed, it became apparent that a fill-type structure was better suited to site conditions than was a concrete structure. Field explorations indicated that most of the required overburden excavation will be suitable materials for the random fill, and that rock excavation in the intake, spillway, and powerhouse areas can be utilized for the rock shell portion of the dam.

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[18] "Furnas Hydroelectric Project," by James W. Libby, Proceedings, ASCE, Vol. 86, No. PO 2, April, 1960.

Because of the slabby nature of the quartzite fragments to be used for the rock shells, it was considered advisable to make special investigations of the strength characteristics of this material. The United States Bureau of Reclamation in Denver was retained to perform large scale triaxial compression tests on the rockfill using high all around pressures.

Suitable earth-core material is available on the right bank about a mile upstream from the dam axis and on the left bank about 4 miles from the site. Adequate transition zone material is available from spillway overburden excavation. Routine tests of core and transition zone materials have been performed in Brazilian laboratories and have determined the relative stability of the material in the various borrow pits. In addition, test strips have been constructed at the site to correlate field and laboratory compaction results.

The canyon at the Furnas site is 350 m (1,150 ft) wide on the axis at river level. Natural ground extends well above maximum reservoir level on both abutments. Opposed to these favorable conditions the foundation in the river bed is traversed by a deep fault that has been closed with a diabase intrusion.

The section of the dam as shown in Fig. 28 was adopted after both circular arc and sliding wedge analyses showed that it was a structurally stable design.

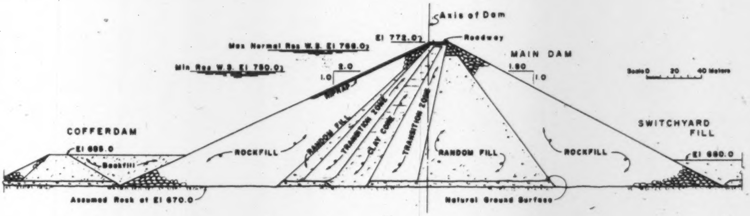
Axis of Dam
El 772.0
Roadway
Max Normal Res W.S. El 768.0
Min Res W.S. El 750.0
2.0
1.0
MAIN DAM
1.80
1.0
Scale 0 20 40 Meters
Riprap
COFFERDAM
SWITCHYARD FILL
El 695.0
Backfill
ROCKFILL
RANDOM FILL
TRANSITION ZONE
CLAY CORE
TRANSITION ZONE
RANDOM FILL
ROCKFILL
El 680.0
Assumed Rock at El 670.0
Natural Ground Surface

FIG. 28.—FURNAS PROJECT, DAM SECTION

Strength parameters for all materials utilized in the stability analysis were determined from a study of the results of laboratory tests to be as follows:

Zone	c, in psi	ϕ, in degrees
Clay core	14	15
Transition zone	0	38
Random fill	0	38
Rock fill	0	38

The core and transition zones are arranged as a compromise between an inclined core and a central vertical core. This arrangement was found to utilize available materials most effectively. Rockfill will be placed in layers and compacted by crawler-type equipment.

Horizontal curvature of the main embankment has been accentuated to improve abutment contacts and minimize the possibility of cracks developing in the core due to downstream movement (Fig. 29). Because of the steepness of the left abutment, special precautions will be taken to ensure that detrimental effects of differential settlement will not endanger the safety of the core. The upstream and downstream transition zones will be extended along the rock face.

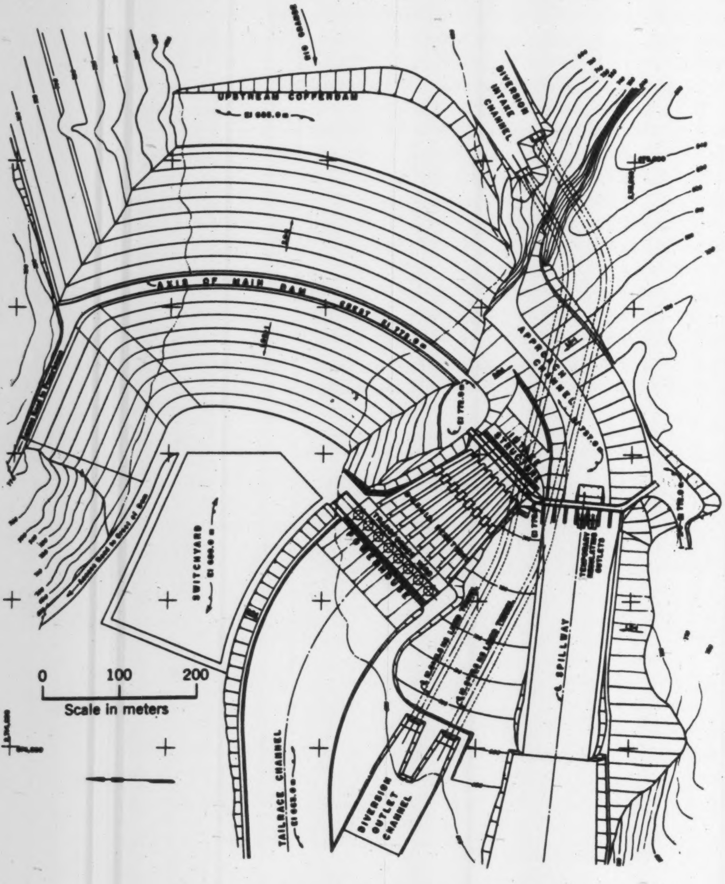
RIO GRANDE
UPSTREAM COFFERDAM
DIVERSION INTAKE CHANNEL
AXIS OF MAIN DAM
CREST
APPROACH CHANNEL
SWITCHYARD
SPILLWAY
TAILRACE CHANNEL
DIVERSION OUTLET CHANNEL
0
100
200
Scale in meters

FIG. 29.—GENERAL PLAN OF F'TRNAS PROJECT

Excavation to suitable foundation on the left abutment is expected to be nominal. The right side will require considerably more excavation to provide a suitable contact area for the clay core against sound-impervious abutment rock. A small amount of gouge and fractured rock excavation in the river channel will expose a suitable foundation.

The writer wishes to express his appreciation of the interest taken by the contributors to the discussion. Active participation of these interested persons has made possible a more comprehensive presentation for inclusion in this Symposium on Rockfill Dams. It is through such activities that significant technical information may be disseminated throughout the world for the benefit of all.

An aerial view of the completed Brownlee project is shown in Fig. 27.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3083

ROCKFILL DAMS: WISHON AND COURTRIGHT CONCRETE FACE DAMS

By J. Barry Cooke,[1] F. ASCE

With Discussion by Messrs. G. I. Davey; Karl Terzaghi; and J. Barry Cooke

SYNOPSIS

Design considerations and construction of the two approximately 300-foot high dams are discussed. All changes in design and construction from previous P. G. and E. dams have been made to lower the cost without affecting the safety of the dams. Wishon was filled in May 1958 and initial crest settlement data is presented.

INTRODUCTION

Wishon and Courtright dams are located on the North Fork watershed of the Kings River about 60 miles from Fresno, California. Together they will store 251,300 acre feet of water which will fall through 5645 feet of head in Pacific Gas and Electric Company's Haas, Balch and Kings River hydroelectric plants.(2,8)[2] Physical data in comparison with the Company's Salt Springs and Lower Bear River dams is presented in Table I, and discussion of the two earlier dams is presented in reference (10).

Wishon Dam was completed on May 10, 1958, and the reservoir completely filled on May 27, 1958. Courtright will be completed in 1958. The two dams were designed concurrently and are therefore similar. Wishon will be reviewed in detail and Courtright will be discussed only as it differs from Wishon.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1746. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

[2] Numerals in parenthesis refer to corresponding items in the Bibliography.

Wishon Design

The selection of the concrete face rockfill type of dam was made on the basis of its being the lowest cost dam considering the site, availability of materials, and reservoir operating conditions. The site is exposed glaciated granite with essentially no overburden or decomposed granite, Fig. 7. Exploration for core materials, for core rockfill types of dam, failed to locate satisfactory materials in meadows or in zones of decomposed granite in a radius of about 20 miles. Reservoir operation will normally unwater the top two-thirds of the face each year and all may be unwatered, if necessary, without impairing operation.

The cost of a concrete gravity dam was estimated at twice that of a concrete face rockfill. A thin central core rockfill and a sloping core rockfill were estimated to cost about the same as a concrete face rockfill. A decision between types of rockfill based on reasons other than economic was not required since core material was not available.

Having selected the concrete face rockfill type of dam, the experience record of Salt Springs, and of the Lower Bear River dams, was thoroughly reviewed. The review covered construction as well as design and performance, with the objective of specifying a dam that could be constructed by modern methods and in a straightforward manner. All changes from Lower Bear River design were made to obtain a more economical design and to simplify construction. The changes do not alter the safety of the dam; they should not adversely affect the performance; and they substantially lowered the cost. The main changes in order of the amount of saving are:

(1) Thinner placed rock.
(2) Steeper upstream face.
(3) Less rigid placed rock specification.
(4) Use of face lifts for placed rock and face flab construction access.
(5) Elimination of keyways in the placed rock.
(6) Permit placed rock points to project to the plane of the reinforcing steel.
(7) Simplified joints.

Diversion, Outlet Works and Spillway

Diversion Tunnel.—The size and alignment of diversion tunnel was given to the bidders. In the first runoff season, the partially completed dam, with broad crest width and no concrete face, was to serve as cofferdam. In the second runoff season, the permanent outlet works was to be installed in order to act as a spillway, to permit storage of water with the partially completed dam. The design capacities were such that several floods of record would have caused not more than 2000 cfs to pass through the partially completed rockfill. It was considered that such would not endanger this dam of large rock, and the economy in size of tunnel and outlet works was obtained. Circumstances were such that actually no flows passed through the rockfill.

Outlet Works.—The capacity was governed by the use of the outlet works as a spillway between the second and third construction season in order to obtain early storage, as discussed above. An 84-inch Howell-Bunger valve and a 120-inch butterfly valve are installed in a chamber in the diversion tunnel, as illustrated and described in reference (8). The diversion tunnel leading to and from the valve chamber is left unlined.

Spillway.—The spillway, Fig. 1, consists of six 40-foot by 11-foot radial gates that discharge into the spillway quarry, an unlined rock channel. The channel discharges onto exposed bedrock and the flow reaches the river a quarter of a mile downstream from the dam. The design flow is 30,000 cfs for the maximum storage and flood level of 6550, and freeboard is provided by the 4-foot coping wall, Fig. 5. The concrete face rockfill dam is not considered to require as much freeboard as an earth or earth core rockfill dam. The tops of five auxiliary concrete dams are only one foot above maximum storage level, and the spillway channel (quarry floor) is sloped to take increased flow without causing backwater effect at the spillway gate structure. These two features give excess spillway capacity at no cost.

Auxiliary Dams

Five concrete gravity auxiliary dams are required along a glaciated granite ridge that forms the right abutment of the main dam. The maximum height is 30 feet and total crest length 240 feet. The crest being 1 foot above maximum storage level and 3 feet below top of coping wall, the auxiliary dams are of minimum cost and provide excess spillway capacity.

Cross Section and Alignment

The cross section of the dam, Fig. 1, is similar to Salt Springs, Table 1, but with slightly steeper upstream face. At Lower Bear River dams the construction experience showed that the 1:1 face of Dam No. 2 could be constructed at essentially the same unit costs as the 1.3:1 face of Dam No. 1. It was considered that the use of "face lifts" proposed for Wishon would make the 1:1 face more easily constructed, and that the reduced quantities would directly represent reduced cost.

Upstream Face.—The upstream face slope, Fig. 1, may be considered to be 1:1 in the upper portion and 1.3:1 in the lower portion of the dam. At elev. 6400, 150 feet from the crest, a transition between the two slopes is made by a 40-foot height of slab at 1.1:1 above elev. 6400 and another at 1.2:1 below. The 1:1 slope is stable with a minimum thickness of placed rock; can be constructed conveniently; and concrete can be placed rapidly by slip forms. The

TABLE 1.—STATISTICAL DATA FOR FIVE P. G. AND E. CO. ROCKFILL DAMS

		SALT	LOWER BEAR RIVER		WISHON		COURT-
	UNIT	SPRINGS	NO. 1	NO. 2	MAIN	WING	RIGHT
Year Completed	YR	1931	1952	1952	1958	1958	1959
Storage	AF	142,000	49,000		128,500		123,300
Crest Elevation	FT	3958	5820		6500		8188
Height(Above Foundation)	FT	332	250	155	296	166	317
Crest Length	FT	1300	960	865	1415	1915	900
Camber (Max.)	FT	6	3.3	1.1	4.0	2.2	4.2
Surface Area of Face	SF	380,000	190,000	90,000	650,000		220,000
Crest Width	FT	15	20	20	20	20	16
Max. Height (To Design Crest)	FT						
Streambed at Axis		302	234	107	256	151	295
Foundation at Axis		328	234	150	260	158	295
Streambed		313	250	125	256	160	317
Foundation		332	250	155	296	166	317
Cutoff Line		318	221	137	250	130	289
Cutoff Trench		348	238	148	288	152	337
Slopes (Horiz. to 1 Vert.)							
Upstream		1.4 to 1.1	1.3	1.0	1.3 to 1.0	1.0	1.3 to 1.0
*Downstream		1.4	Nat. to 1.4	Nat.	Nat.to 1.4	Nat.	Nat.to 1.4
Placed Rock Thickness	FT						
Top		15	10	10	7.8	7.8	7.8
Down 100 Ft		15	15	15	7.8	7.8	7.8
Down 150 Ft		15	17.5	-	10	-	10
Down 200 Ft		15	20	-	10	-	10
Max. (Cutoff Line)		15	21	16.9	11.6	7.8	11.6
Quantities (1000 CY)							
Excavation		312	10	138	270		60
Rock-Dumped		2,744	900	349	3,530		1,540
Placed		224	89	43	200		77
Total		2,968	989	392	3,730		1,617
Placed:Dumped	%	8.1	9.9	12.0	5.7		5.0
**Concrete		45	23	11	86		38

* Nat. = Natural = 1.3 Plus

** Includes cutoff, coping, face slab theoretical and excess thickness, and abutment gravity sections.

1.3:1 slope was preferred below a height of 150 feet to transmit the higher water pressures more directly to the foundation through a shorter distance in the rockfill; to better transmit the horizontal component of water pressure to the foundation in the vicinity of the cutoff wall; and to show increased respect for the higher dam. Adjacent to the cutoff, the rocks underlying the slab must transmit the horizontal component of water pressure to bedrock with minimum yielding or movement. With the 1.3:1 face slope, this is accomplished with a ratio of vertical to horizontal force of 1.3:1, which is more favorable than 1:1. The 1.3:1 slope reduces face movements in the heavily loaded lower portion of the dam, by transmitting the water load to the foundation through a shorter distance in dumped rock. The cost of the flatter slope is not great since it occurs in the narrow portion of the site. The full height of the wing section of maximum 166-foot height, Fig. 1 and 6, is at 1:1 upstream slope. The terms "wing section" and "main section", Fig. 1, are used to identify the two rather independent portions of Wishon Dam.

Downstream Slope.—The downstream slope is essentially the natural dumped slope. Because the natural slope is irregular and unknown, the first lifts are carried out to a toe located by a 1.4:1 slope from a 12-foot wide theoretical crest at the storage level of 6550. The upper portion of the slope is the natural slope that results from constructing the 20-foot crest width to the constructed crest elevation, Fig. 1 and 5. The 20-foot crest width provides for two-way traffic and otherwise is wider than considered necessary for the concrete face rockfill dam.

Alignment.—The horizontal alignment of the dam is based largely on determining minimum quantities and cost. For the concrete face dam, a location giving minimum face area, rather than minimum volume, tends to govern. It is noted, Fig. 1, that the cutoff is along the ridge on both abutments, which requires more dumped fill, but gives a dam having minimum face area and minimum cost. The dam is on a 5000-foot radius curve. The face joints of the main and wing sections are normal to two chords that intersect at Station 15 + 00, Fig. 2. The curvature is somewhat more than required for settlement and appearance in order to obtain minimum cost, by adapting the long dam to the site contours. The vertical alignment provides a crown four feet above design crest at the 296-foot high maximum section.

Cutoff

Abutment Gravity Sections.—At each end of the dam a concrete gravity section, Fig. 2, is used up to a height of 30 feet, at about which height the cost of a gravity section equals that of the concrete face rockfill. The upstream face is 1:1 and, with copper joint, is connected to the first face slab of the rockfill dam. The use of this gravity section provides economical and straightforward ends for the dam. The coping wall continues, dowelled to bedrock, until it is of zero height.

Concrete Face Cutoff.—The cutoff is shown on Fig. 3 in comparison with the cutoff of Salt Springs and Lower Bear River dams. It is discouraging to excavate sound rock and replace it with concrete, and the trend has been to reduce the depth of cutoff in sound bedrock. In seamy rock the cutoff is carried to greater depths. Even with careful blasting, there is a tendency to loosen adjacent rock and the cost of the excavation is very high. The trench is upstream from the cutoff line to eliminate uplift. If inferior rock were encountered in local areas it was intended to provide an upstream slab or to

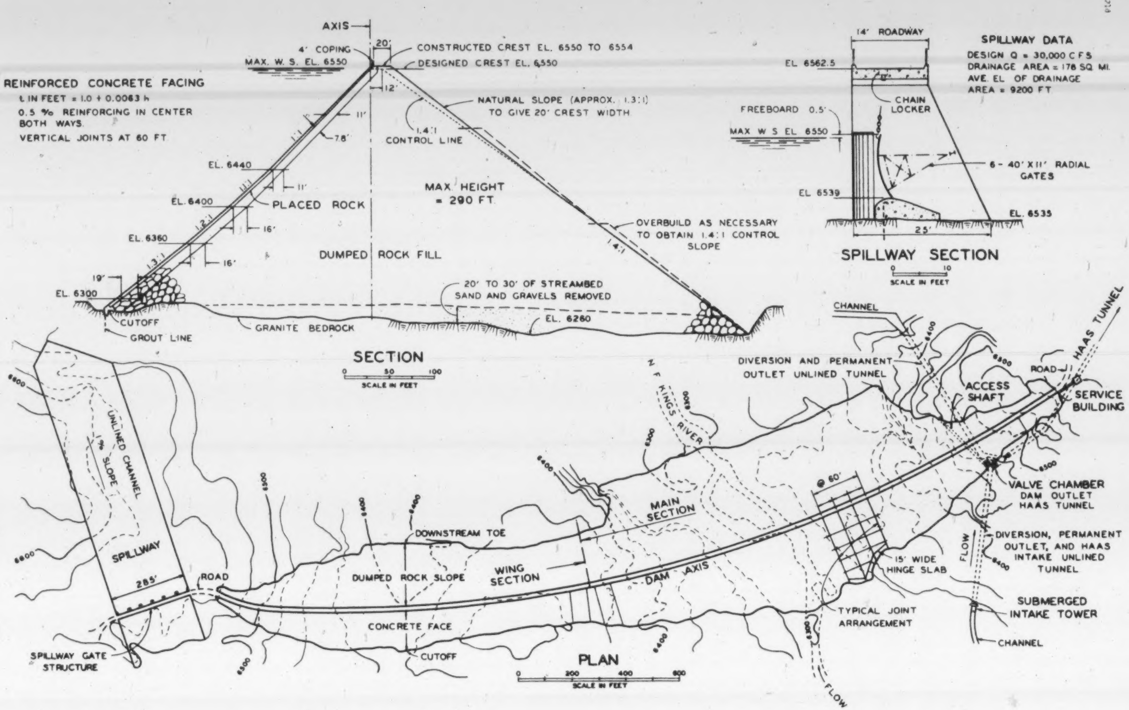

AXIS
4' COPING
MAX. W. S. EL. 6550
20'
CONSTRUCTED CREST EL. 6550 TO 6554
DESIGNED CREST EL. 6550
12'
REINFORCED CONCRETE FACING
t IN FEET = 1.0 + 0.0063 h
0.5 % REINFORCING IN CENTER BOTH WAYS.
VERTICAL JOINTS AT 60 FT.
NATURAL SLOPE (APPROX. 1.3:1) TO GIVE 20' CREST WIDTH
1.4:1 CONTROL LINE
11'
7.8'
1:1
EL. 6440
11'
EL. 6400
1.1:1
PLACED ROCK
16'
1.2:1
EL. 6360
16'
1.3:1
19'
EL. 6300
CUTOFF
GROUT LINE
GRANITE BEDROCK
MAX HEIGHT = 290 FT
DUMPED ROCK FILL
20' TO 30' OF STREAMBED SAND AND GRAVELS REMOVED
EL. 6280
OVERBUILD AS NECESSARY TO OBTAIN 1.4:1 CONTROL SLOPE
1.4:1
SECTION
0 50 100
SCALE IN FEET
14' ROADWAY
EL. 6562.5
CHAIN LOCKER
FREEBOARD 0.5'
MAX W S EL 6550
6 - 40' X 11' RADIAL GATES
EL 6539
EL. 6535
25'
SPILLWAY SECTION
0 10
SCALE IN FEET
SPILLWAY DATA
DESIGN Q = 30,000 C F S
DRAINAGE AREA = 178 SQ. MI.
AVE. EL. OF DRAINAGE AREA = 9200 FT
CHANNEL
N F KINGS RIVER
DIVERSION AND PERMANENT OUTLET UNLINED TUNNEL
ROAD
HAAS TUNNEL
ACCESS SHAFT
SERVICE BUILDING
VALVE CHAMBER
DAM OUTLET
HAAS TUNNEL
DIVERSION, PERMANENT OUTLET, AND HAAS INTAKE UNLINED TUNNEL
FLOW
SUBMERGED INTAKE TOWER
CHANNEL
@ 60'
15' WIDE HINGE SLAB
TYPICAL JOINT ARRANGEMENT
MAIN SECTION
WING SECTION
DAM AXIS
DOWNSTREAM TOE
DUMPED ROCK SLOPE
CONCRETE FACE
CUTOFF
UNLINED CHANNEL
1% SLOPE
SPILLWAY
285'
ROAD
SPILLWAY GATE STRUCTURE
6600
6500
6400
6300
FLOW
PLAN
0 200 400 600
SCALE IN FEET

FIG. 1.—PLAN, SECTION, AND SPILLWAY SECTION OF WISHON DAM

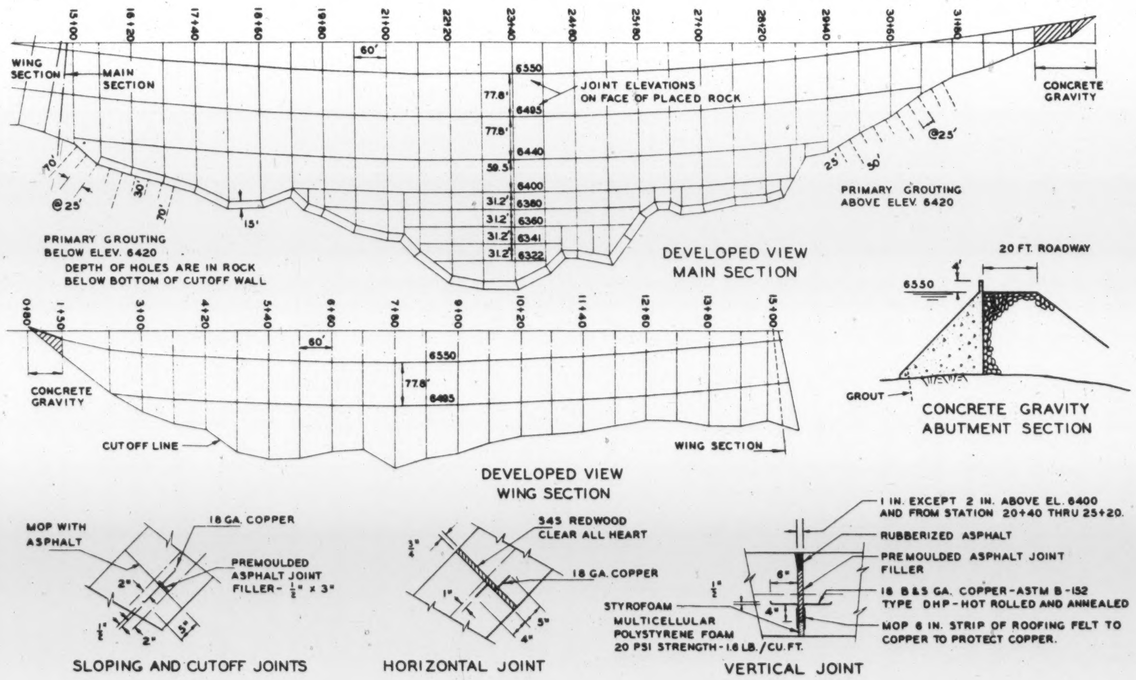
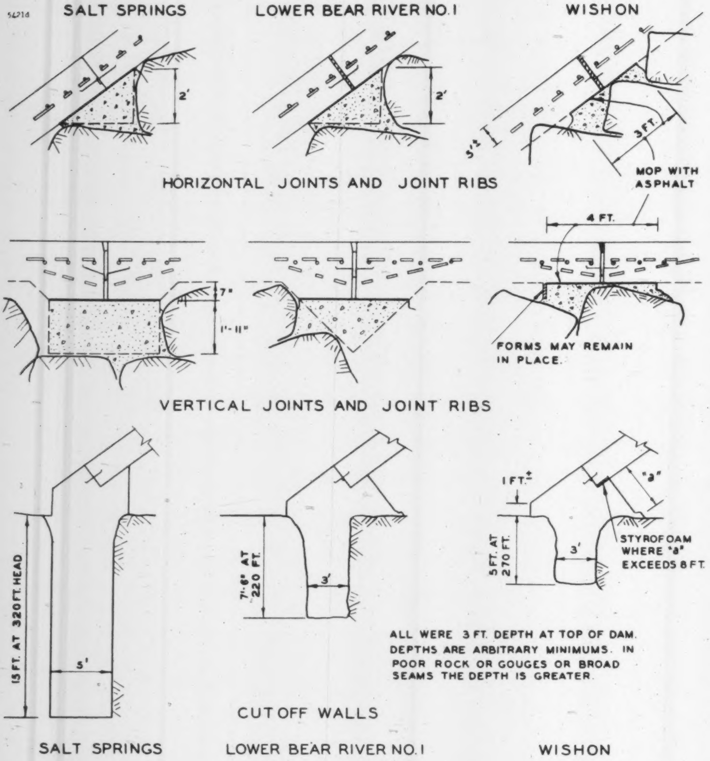

15+00
16+20
17+40
18+60
19+80
21+00
22+20
23+40
24+60
25+80
27+00
28+20
29+40
30+60
31+80
60'
WING SECTION
MAIN SECTION
6550
77.8'
6495
JOINT ELEVATIONS ON FACE OF PLACED ROCK
77.8'
6440
59.5'
6400
31.2'
6380
31.2'
6360
31.2'
6341
31.2'
6322
CONCRETE GRAVITY
@25'
25'
50'
PRIMARY GROUTING ABOVE ELEV. 6420
70'
30'
70'
@25'
15'
PRIMARY GROUTING BELOW ELEV. 6420
DEPTH OF HOLES ARE IN ROCK BELOW BOTTOM OF CUTOFF WALL
DEVELOPED VIEW MAIN SECTION
0+80
1+30
3+00
4+20
5+40
6+60
7+80
9+00
10+20
11+40
12+60
13+80
15+00
60'
6550
77.8'
6485
CONCRETE GRAVITY
CUTOFF LINE
WING SECTION
DEVELOPED VIEW WING SECTION
20 FT. ROADWAY
4'
6550
GROUT
CONCRETE GRAVITY ABUTMENT SECTION
MOP WITH ASPHALT
18 GA. COPPER
PREMOULDED ASPHALT JOINT FILLER - 1/2" x 3"
2"
1/2"
2"
5"
SLOPING AND CUTOFF JOINTS
3/4"
S4S REDWOOD CLEAR ALL HEART
18 GA. COPPER
1"
5"
4"
HORIZONTAL JOINT
1 IN. EXCEPT 2 IN. ABOVE EL. 6400 AND FROM STATION 20+40 THRU 25+20.
RUBBERIZED ASPHALT
PREMOULDED ASPHALT JOINT FILLER
6"
1/2"
4"
18 B&S GA. COPPER - ASTM B-152 TYPE DHP - HOT ROLLED AND ANNEALED
STYROFOAM MULTICELLULAR POLYSTYRENE FOAM 20 PSI STRENGTH - 1.6 LB./CU. FT.
MOP 6 IN. STRIP OF ROOFING FELT TO COPPER TO PROTECT COPPER.
VERTICAL JOINT

FIG. 2.—JOINT ARRANGEMENT AND DETAILS, GROUTING PATTERN, ABUTMENT GRAVITY SECTION, WISHON DAM



SALT SPRINGS LOWER BEAR RIVER NO. I WISHON

HORIZONTAL JOINTS AND JOINT RIBS

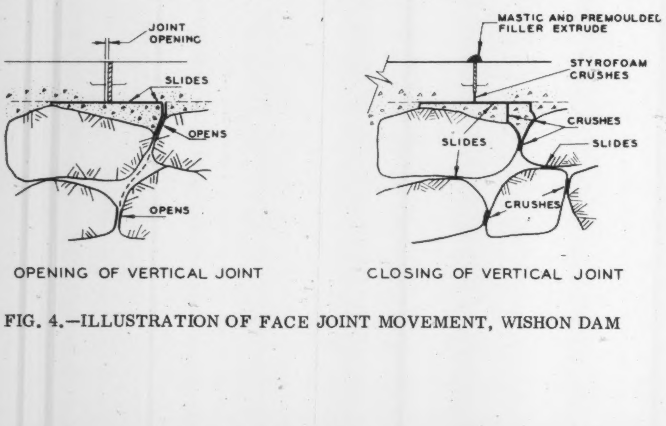
VERTICAL JOINTS AND JOINT RIBS

CUTOFF WALLS

SALT SPRINGS LOWER BEAR RIVER NO. I WISHON

56216
2'
2'
5"±
3 FT.
MOP WITH ASPHALT
4 FT.
7"
1'-11"
FORMS MAY REMAIN IN PLACE.
1 FT.±
"a"
STYROFOAM WHERE "a" EXCEEDS 8 FT.
5 FT. AT 270 FT.
3'
7'-6" AT 220 FT.
3'
15 FT. AT 320 FT. HEAD
5'
ALL WERE 3 FT. DEPTH AT TOP OF DAM. DEPTHS ARE ARBITRARY MINIMUMS. IN POOR ROCK OR GOUGES OR BROAD SEAMS THE DEPTH IS GREATER.

FIG. 3.—COMPARISON OF CUTOFF AND JOINT RIBS, P. G. AND E. DAMS



OPENING OF VERTICAL JOINT CLOSING OF VERTICAL JOINT

JOINT OPENING
SLIDES
OPENS
OPENS
MASTIC AND PREMOULDED FILLER EXTRUDE
STYROFOAM CRUSHES
CRUSHES
SLIDES
SLIDES
CRUSHES

FIG. 4.—ILLUSTRATION OF FACE JOINT MOVEMENT, WISHON DAM

bed the lower layer of placed rock in concrete to obtain a longer percolation path, but this was not required. Across some seamy or gouged zones, the cutoff wall became as high as 15 feet and styrofoam was used, Fig. 3,to allow the face to yield with the placed rock adjacent to the cutoff wall.

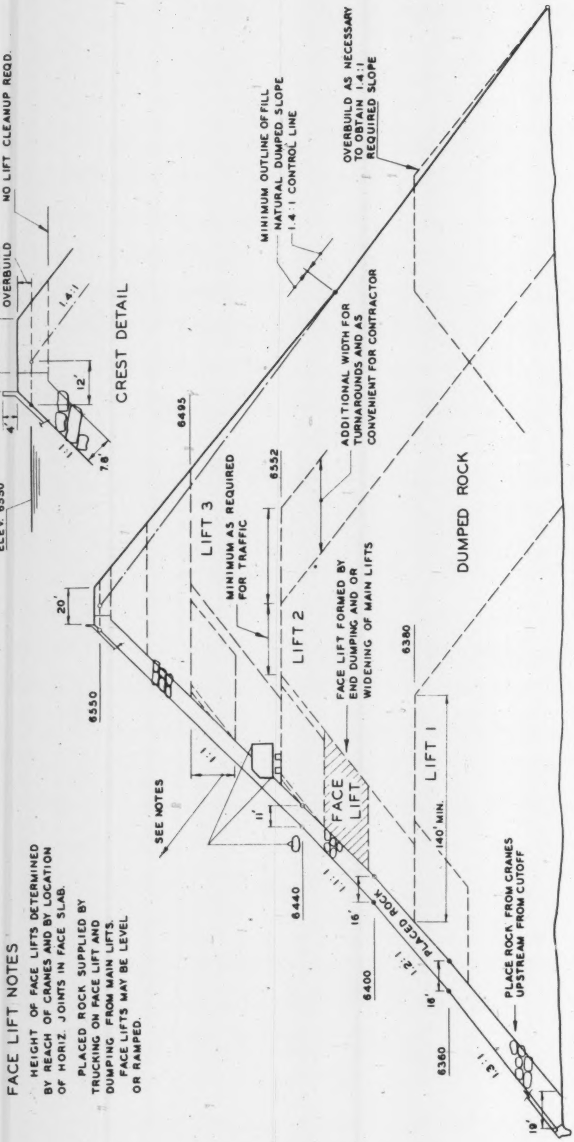
Grouting.—Grout pipes were arbitrarily placed in the cutoff wall normal to the cutoff line and at 25-foot spacing, and additional pipes located so that a drill hole will intersect visible seams some 5 to 10 feet below the cutoff trench. The arbitrary pattern of grouting is shown on Fig. 2. The deeper holes at 100-foot spacing were cored and were water pressure tested before grouting. The requirement was for additional holes and regrouting as determined by the grout take from the primary holes.

Dumped Rockfill

Specification.—Selection of quarry sites was left to the contractor, except for specific excavation requirements in the spillway area, and with the restriction that no quarry be closer than 200 feet to the cutoff line or outlet tunnel if the quarry floor is below the crest of the dam. To assure large size rock in the fill, the rock handling equipment was specified to be capable of handling 15-ton rocks. Rock was specified to be sound, hard and durable with definitions for each term. At least 50% (by weight) of the rock in the dam was specified to range from 0.5 to 10 ton and over.

Lifts.—Maximum height of lifts are considered desirable both to the performance of the dam and to the contractor. The high lifts are associated with maximum energy and compaction going into the fill during construction. The high lifts require minimum haul road construction and minimum lift surface area to be cleaned up. A proposed, acceptable, but not required, lift arrangement, Fig. 5, was included in the bidding drawings to give the contractors an idea of one way the dam could be constructed in a straightforward manner. Lift 1, Fig. 5, was specified at elev. 6380 to permit early placed rock construction and provide a cofferdam by the end of the first construction season. The 140-foot minimum top width was to minimize upstream face movement in the event of overpour while this lift served as a cofferdam. Lift 2 was at an elevation that would assure completion across the canyon by the time the placed rock was completed to elev. 6380, and thus permit continuous work on all components of the dam. Lift 3 is at a required elevation to assure partial storage of 80,000 acre feet at the end of the second year. The three main lifts are set back from the face to permit "construction access" or "face" lifts. The top of each lift is shown at minimum construction widths. The minimum width, if used, would require minimum lift surface cleanup and give maximum compaction to the dam by the high overlapping fill. However, the benefits are not essential to the dam and broader widths are permitted if they facilitate construction.

The face lifts are theoretically undesirable because the low lifts do not get as much settlement during construction and the placed rock and concrete are constructed on the freshly dumped fill of the face lifts. However, since these considerations only affect the probability of cracks in the face, and not safety, the method is adopted because of the practical and lower cost construction that results in the use of face lifts. The face lifts may be constructed rapidly since the area of the "parallelogram" section of the lift is small. Since there is only a small amount of dumping from the end of the face lift as it progresses, not many fines accumulate and lift surface cleanup is not a problem. The



FACE LIFT NOTES

HEIGHT OF FACE LIFTS DETERMINED BY REACH OF CRANES AND BY LOCATION OF HORIZ. JOINTS IN FACE SLAB.

PLACED ROCK SUPPLIED BY TRUCKING ON FACE LIFT AND DUMPING FROM MAIN LIFTS. FACE LIFTS MAY BE LEVEL OR RAMPED.

DESIGN CREST
MAX. STORAGE W.S.
ELEV. 6550
20'
5'
4.0' MAX. OVERBUILD
CONSTRUCTION LEVEL FOR PLACING ROCK. NO LIFT CLEANUP REQD.
1.4:1
12'
4'
1:1
7.8'
CREST DETAIL
20'
6550
6495
6552
6380
6440
6400
6360
LIFT 3
LIFT 2
LIFT 1
FACE LIFT
DUMPED ROCK
PLACED ROCK
SEE NOTES
1:1
11'
16'
16'
10'
1.1:1
1.2:1
1.3:1
140' MIN.
MINIMUM AS REQUIRED FOR TRAFFIC
FACE LIFT FORMED BY END DUMPING AND OR WIDENING OF MAIN LIFTS
ADDITIONAL WIDTH FOR TURNAROUNDS AND AS CONVENIENT FOR CONTRACTOR
MINIMUM OUTLINE OF FILL NATURAL DUMPED SLOPE
1.4:1 CONTROL LINE
OVERBUILD AS NECESSARY TO OBTAIN 1.4:1 REQUIRED SLOPE
PLACE ROCK FROM CRANES UPSTREAM FROM CUTOFF

FIG. 5.—PROPOSED CONSTRUCTION METHOD, MAIN SECTION, WISHON DAM

height and location of face lifts is as governed by the location of concrete joints and by the reach of the cranes for placing rock.

Sluicing.—Sluicing water was specified to be applied at the times and places where rock is being dumped in a volume three times as great as the volume of rock being dumped. Sluicing is also required, as may be directed, on visible surface zones of fines that may develop. A pressure of 100 psi at the nozzles of 2 to 3-inch diameter was required, which produced jets of 2.5 to 5.5 cfs. A pressure gage was required at each monitor. The action of the powerful jets in excavating pockets of spalls and distributing the spalls into voids of the large rocks is considered as desirable a feature of sluicing as the washing of small fines into the rockfill.

Placed Rock

The placed rock section at Wishon is somewhat thinner than used on previous high rockfill dams of the P. G. and E. Company. It is considered that the thickness does not involve stability for the slopes of 1:1 or flatter than 1:1; and that, with the quality of the well sluiced dumped fill, the thickness should have little effect upon the probability of cracks in the face. The 11-foot horizontal thickness (7.8-foot normal thickness) in the upper portion of the dam, Fig. 1, was adopted as a practicable construction minimum to catch the dumped rocks with the steeper than natural upstream surface slope. However, in the lower portion of the dam, the placed rock was increased to about 11-foot normal thickness, in spite of the flatter face slopes. This is still considerably thinner than at Salt Springs and Lower Bear River, but is thicker than the 7.8-foot minimum adopted for the upper portion of the dam out of respect for the high water pressures in the lower portion of a 300-foot dam, Table I. Since only a small per cent of the surface area of the dam is in the lower portion of the dam, the increased thicknesses at the bottom of Wishon require only 9.5% more placed rock than if all the placed rock were of 11-foot horizontal thickness. It was decided not to depart too much from previous practice at this time. Since the thickness and specification for placed rock is considered to involve economics and practical construction, and not safety, the construction and performance experience at Wishon may lead to further economies in the future.

Specification.—The drawings, Fig. 1 and 6, specify horizontal thicknesses, since horizontal thicknesses can best be used to lay out and inspect the construction. For the wing section, Fig. 6, the 11-foot thickness is used for the full height of dam. The rock was specified to be selected rock, 1 to 10 ton or more in weight, with average weight of not less than 3 tons. Rocks were required to touch each other in up and downstream direction and be as close as possible laterally, with nearest points of adjacent rock not greater than 6 inches apart. Accessible voids between up and downstream rocks were required to be chinked with large rock, and lateral voids to be filled with rocks that would pass a 2-inch grizzly. Rock points or edges were permitted to project to the plane of the reinforcing steel to minimize the amount of excess concrete beyond the theoretical thickness.

In previous specifications for placed rock, the requirement of lateral as well as up and downstream contact of the placed rocks greatly slowed down production. In writing the Wishon Specifications, it was considered that the possible benefits were not worth the cost, and lateral contact was not

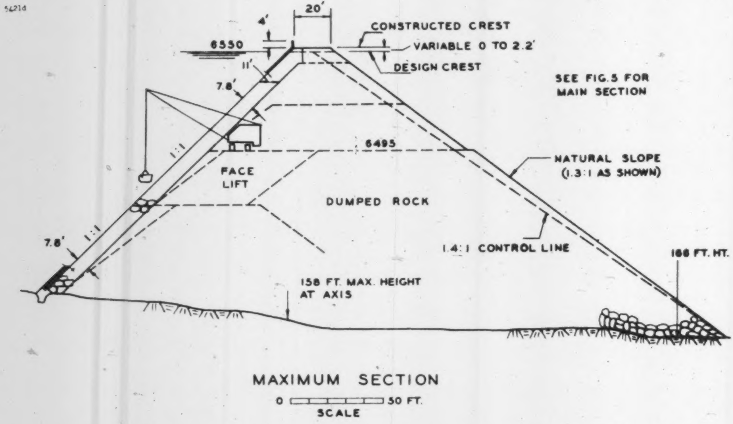

54214
20'
4'
CONSTRUCTED CREST
6550
VARIABLE 0 TO 2.2'
DESIGN CREST
11'
7.8'
SEE FIG.5 FOR MAIN SECTION
1:1
6495
NATURAL SLOPE (1.3:1 AS SHOWN)
FACE LIFT
DUMPED ROCK
7.8'
1:1
1.4:1 CONTROL LINE
168 FT. HT.
158 FT. MAX. HEIGHT AT AXIS
MAXIMUM SECTION
0
50 FT.
SCALE

FIG. 6.—PROPOSED CONSTRUCTION, WING SECTION, WISHON DAM

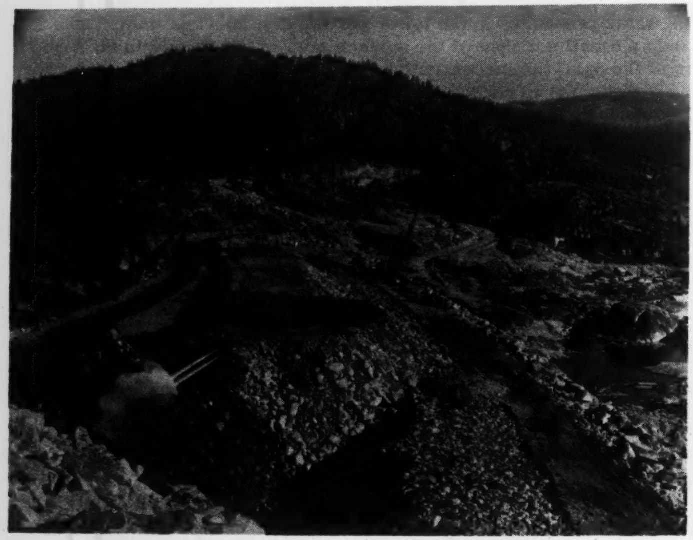


FIG. 7.—KINGS RIVER PROJECT WISHON DAM, DUMPING ROCK NEAR THE CENTER OF THE DAM, CONSTRUCTION INVOLVES HAULING 3,687,000 CU YD OF DUMPED AND PLACED ROCK IN 17-CU YD TRUCKS, DOUBLE MONITOR IS OPERATED BY ONE MAN

required. This change involved definite capital cost versus "possible" maintenance cost, and not safety. Vertical contacts are important and occur automatically. Up and downstream contacts transmit the high water pressure into the fill with minimum possibility of local movements.

Concrete Face

The concrete face, at elevation 6550 in the Sierras, will be subject to freezing conditions. The strength is specified on the basis of obtaining a dense frost action resistant concrete and not to resist computed stresses. Concrete was specified to have: 3000 psi 28-day compressive strength; 2-1/2 inch maximum size aggregate; 5-1/2 to 6% air entrainment by volume; and contain a water reducing agent. Cold weather pouring requirements were: not less than 40° F placing temperature in moderate weather; and not less than 50° F temperature for 72 hours after placing in freezing weather.

Thickness and Reinforcing.—The thickness of concrete and the reinforcing, Fig. 1, are the same as at Salt Springs and Lower Bear River. It is arbitrary and has been satisfactory at those dams. One economy in thickness, made at Wishon, was to permit placed rock points and edges to project into the theoretical thickness of slab to the plane of the reinforcing steel. This reduces the amount of concrete beyond the theoretical thickness that is required to fill the surface irregularities of the placed rock.

Joint Arrangement.—The joint arrangement, Fig. 2, is simplified over that used at Lower Bear River to improve performance and at the same time use a minimum total length of joints. The 60-foot vertical joint spacing is used as a practicable spacing for use of a slip form and to give reasonable alignment and flexibility at the cutoff. The wing section, of 166-foot maximum height has only one horizontal joint that divides the face into two pours. In the main section the top two slabs are of 77.8-foot lengths, selected as practical lengths for maximum single pours. The slabs below elevation 6400 are 31.2 feet long because in the lower half of the face the settlement contours are closer together; there is a greater tendency to develop vertical stresses in the face; joint movements are high; and the slabs are thicker and more rigid. The joint that is parallel to, and 15 feet from, the cutoff joint is called a sloping joint. It forms a "hinge slab". The sloping joint is used only where the dam exceeds a 150-foot height since above that point the abutment slopes are gradual at the Wishon site and the movements should be small. In the lower portion, additional vertical joints are used at abrupt changes in cutoff alignment. In general, the lower portion of the dam is given more careful joint treatment since cracks can produce more leakage under the high head and the lower face area will seldom be unwatered.

Joint Ribs.—The term "joint rib" refers to the concrete poured in a strip along face slab joint lines in advance of pouring the face slab. The design adopted for Wishon requires no keyway in the rock and is shown in comparison with Salt Springs and Lower Bear River Dams on Fig. 3. At Wishon there are 20,000 feet of joint, excluding the cutoff joint, and the adopted Wishon design was estimated to cost some $500,000 less than the rib designs previously used on this type of dam. The design was adopted to lower the cost, to speed up the placing of rock, and to improve the functioning of the joints. Keyways have slowed up the placed rock work on previous dams and, due to the irregular sized rock, have required excessive concrete.

The joint rib is essentially a smooth surface of concrete painted with asphalt and located under the joint. It is useful both in construction of the dam and in the performance of the joint. The rib gives a surface on which the joint can be conveniently, practically, and well constructed, and on which the forms may be supported. The trim contact of joint rib concrete and the form insures a good concrete job beneath the copper seal. Though there are construction benefits from the rib the primary reason for the rib is its function in performance of the joints. The rib makes the opening and closing of a joint possible without damage to the edges of the slabs by separating the edges of the slab from the placed rock. It automatically provides a secondary waterstop in the vicinity of the joint, but this is not a reason for the joint rib.

Fig. 4 illustrates the functioning of a joint. In the opening of a joint, the placed rock must separate under or adjacent to the joint rib. It can occur anywhere under the rib since the slab can slide on the smooth asphalt surface of the rib. It will probably occur as an irregular separation and the rib concrete would probably crack over the underlying separations in the placed rock. In the closing of a joint, some crushing must occur in the rib or in the immediately adjacent placed rock. Some of the crushed material must find a void in the placed rock, compress pieces of the wood form that are left in place, or compress the styrofoam. The rib is of 1500 psi concrete so that it will crush more readily than the face slab. Below the rib some rock points must crush. If the contact is particularly good between rocks immediately under the joint, some sliding of rocks and crushing of points away from the joint can accommodate the movement.

It is apparent that a substantial force is necessary to close a joint, but experience shows that the joint does close before the slab crushes. That the joints resist closing is probably very desirable since it limits movements. At Lower Bear River Dam No. 2 there has been zero joint opening or closing, which indicates that settlement and temperature forces on the face of this small dam have not been great enough to cause movement.(10) On the higher No. 1 dam, nearly all joints have moved.

Joint Details.—There are three types of joints; sloping and cutoff, horizontal, and vertical. Sloping joints are those parallel to and 15 feet from the cutoff joint, and vertical joints are those that go up the face in a vertical plane perpendicular to the axis. Fig. 2 shows the details of the joints and Fig. 5 shows the setting of the horizontal and vertical joints on the joint ribs. The copper is joined by one-half inch laps that are brazed by a thin flowing, low temperature welding alloy rod, flux being applied on the inner surfaces of the lap. This "sweated" joint is more rapidly made and more easily made watertight than a high temperature brazed job.

The cutoff and sloping joints are cold joints with no ability to close. Near the cutoff line there is so little rock just below the slab that it cannot move very much. It is considered that these joints should relieve moments in the slab but otherwise resist movement of the slabs and support the upper slabs. The redwood is used in horizontal joints as a material that will resist the closing of the joint and not extrude. At Lower Bear River(10) all joints except the upper one closed moderately. It is probable that the redwood is not necessary in the elev. 6495 joint, but it is nominal in cost and all horizontal joints were made the same. The use of the Z waterstop in the horizontal joints is an economy over the use of a U waterstop as on previous dams.

The vertical joint is designed to open or close and to permit the slabs to move parallel to the joint with respect to each other. The deep 4-inch U permits larger opening than will occur and permits the copper to wrinkle rather than tear due to differential movements parallel to the joint. The styrofoam beneath the copper permits complete closing since it compresses to essentially zero volume. The asphalt above the copper can extrude to the surface. Above elev. 6400 the vertical joints of the 8 central vertical joints of the main section were constructed 2 inch open and all other vertical joints one inch open. It is probable that a more simplified vertical joint would be suitable for the wing section based on the performance of the Lower Bear River No. 2 dam(10) but because of the substantial changes in placed rock, the vertical joint detail was not simplified for the wing section at Wishon.

Wishon Construction

The Wishon dam, located at elevation 6550, is snowed in during the winter. A seven month normal construction season may be lengthened to about nine months by fighting snow at both ends of the construction season. In August of 1955 the job was awarded to a joint venture, Morrison-Walsh-Perini, sponsored by Morrison-Knudsen Company. In the remaining several months of the 1955 season, construction started on roads, clearing, opening of quarry and driving of diversion tunnel. Construction in 1956 was primarily streambed excavation, dumping of rock and cut off excavation. The major construction year was 1957, at the end of which Morrison-Knudsen was ahead of schedule, and the remaining work consisted of 40 of the top concrete slabs. It was arranged to complete the dam in winter weather of April-May of 1958 in order to completely fill the 128,000 acre foot reservoir with the 1958 snowmelt runoff. Construction took 21 construction months for the dam of 3350 foot crest, 290 foot maximum height and 3,700,000 cubic yards of rock.

The construction of both Wishon and Courtright is well discussed and illustrated, particularly in regard to methods and equipment, in the construction magazine references (1) (3-7) (9). In this paper construction will be reviewed and illustrated to supplement the more detailed coverage of the references.

Fig. 7 shows the massive granite formation at the site and an interesting stage of construction. The photo is taken from the end of the wing section dumped fill, some of which is in the left foreground. Dumping is proceeding at the 6450 lift, which was carried to the full width of dam rather than partial width as shown on Fig. 5. Placed rock work is proceeding from the face lifts, which were used essentially as proposed on Fig. 5. In the right foreground a portion of the face lift has been scarified to receive the next lift of dumped rock. The pile of fines was formed by monitor and bulldozer in the scarifying operation, and was later removed. From the main lifts the scarified fines are cast over the downstream face. Scarifying is by a bulldozer carrying a monitor that operates at 200 psi. This high pressure is possible by operating at off-shift times when other monitors are shut down. Large rocks may be seen high up on the dumped slopes, which is desirable. It takes well interlocked surface rocks to stop the large rock from sliding further down the slope.

Fig. 8 shows sluicing from cantilevered monitors, and also typical dumped fill. The monitors are located such that they can excavate surface zones of fines as dumping proceeds. Large rocks sliding down the slope cause thin surface slides or movement of rocks, but such surface movements do not endanger the truck rear wheels which are at the edge of the fill. The sluicing operation requires continual effort by inspectors and monitor operators to get the maximum benefit out of sluicing. Inspection of the rockfill shows that the

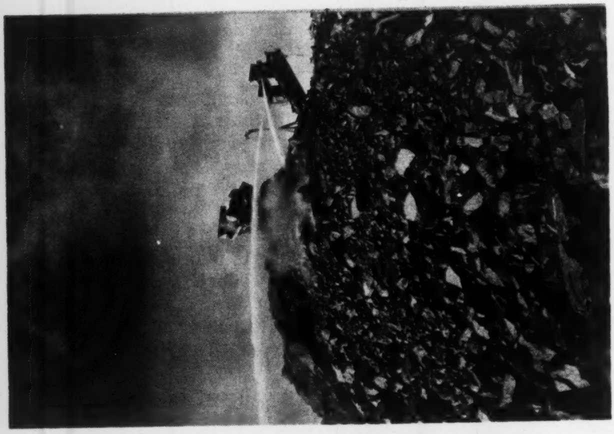


FIG. 8.—TYPICAL ROCKFILL, SLUICING, WISHON DAM

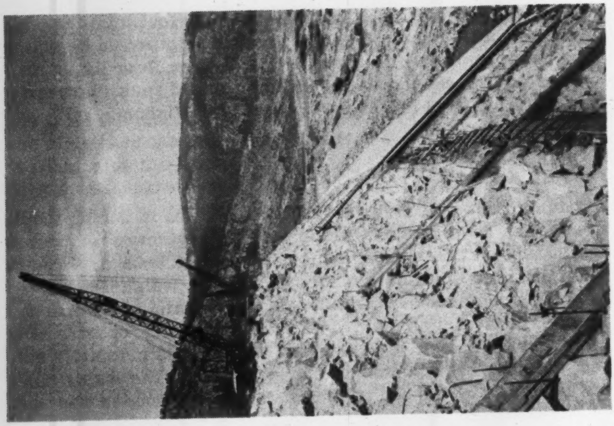


FIG. 9.—FACE CONSTRUCTION, WISHON DAM

rocks come to rest generally in secure-wedged contacts rather than point contacts. The sliding or rolling of the heaviest rocks appears to give material compaction effort to the fill.

Fig. 9 shows the surface placed rock joint ribs and some completed slabs. The placed rocks range in weight from 6 to 15 tons and in maximum dimension, 6 to 12 feet. The surface of the placed rock is quite irregular since the cost of excess concrete in the irregularities is less than the cost of obtaining a smoother face surface with irregular sized and shaped rocks. The small rocks are to fill the surface voids to prevent loss of concrete or concrete mortar in the placed rock. The joint ribs are screeded on the surface, not formed. Concrete may run out under the forms, the only requirement being a surface of 4 foot width for vertical joints and 3 foot for horizontal joints, Fig. 3. The form seen at the edge of the completed slab is reusable and is heavily constructed to carry the wheels of the slip form.

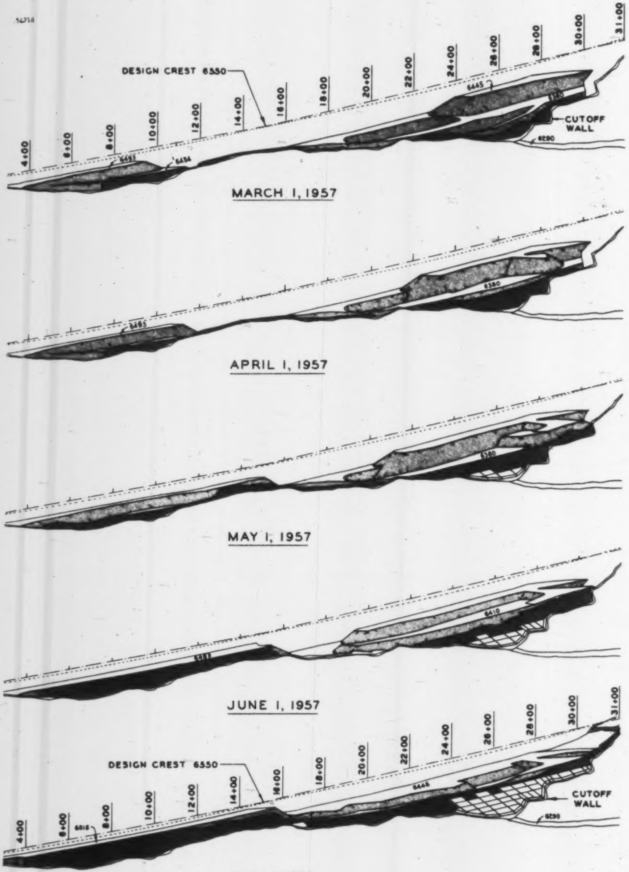
Figs. 10 and 11 show the progress during the 9 months of construction in 1957. These isometric drawings have been redrawn from construction planning and progress drawings made by Morrison-Walsh-Perini. It is seen that construction of the three major elements of the dam, dumped rock, placed rock and concrete face, all proceed simultaneously and continuously. The first isometric shows the work accomplished in six months of 1956, which included primarily the streambed excavation, cutoff excavation, and the dumped rock as shown.

In November of 1957 a small crew was maintained during the cold weather to complete all the dumped and placed rock. The required scheduled completion was only to elevation 6495. Figs. 12 and 13 show two general views of construction stages.

Fig. 14 shows the dam nearing completion in April, 1958. In the middle of March, 1958 an Agreement was signed guaranteeing completion of the dam by June 1, 1958, just in time to catch the snow melt runoff. The dam was snowed in, and as snow was removed and work began a series of storms in late March and early April impaired progress. In spite of the discouraging start the dam was finished on May 9, three weeks ahead of the accelerated schedule. The slip form was designed by Morrison-Knudsen Company. It was operated by electric winches on the form and pulled to blocks anchored to large rocks near the top of the pour.

Foundation Excavation.—The streambed material was sand and gravel with cobbles of 4 to 6 inch size, but there were very few boulders. It was decided to remove them because of their small size and they were removed by dragline working under water. Excavation stopped when the dragline began hitting high spots of bedrock. The holes were not unwatered or completely cleaned out. The estimate, based on several drill holes at and upstream from the axis, was 75,000 cu. yds., but 170,000 cu. yds. were removed and most downstream from the axis. Had this been known more consideration would have been given to leaving most of the material downstream from the axis in place with the use of filters.

Cutoff.—It is considered desirable to make the cutoff line straight between vertical joints in order that the slab may be considered to "hinge" at the cutoff joint. High points of the irregular shaped abutment rock were removed down to the cutoff line, and then the 3 foot wide trench was carefully excavated. The cutoff requires drill holes at about 1 foot centers in both directions, light blasting, and hand mucking. The grouting was carried out to the arbitrary pattern, Fig. 2, with additional holes where seams or jointing planes


DESIGN CREST 6350
4+00
6+00
8+00
10+00
12+00
14+00
16+00
18+00
20+00
22+00
24+00
26+00
28+00
30+00
31+00
6495
6434
6445
6380
CUTOFF WALL
6290
6410
6515

MARCH 1, 1957

APRIL 1, 1957

MAY 1, 1957

JUNE 1, 1957

JULY 1, 1957

FIG. 10.—MONTHLY CONSTRUCTION PROGRESS ISOMETRICS, WISHON DAM

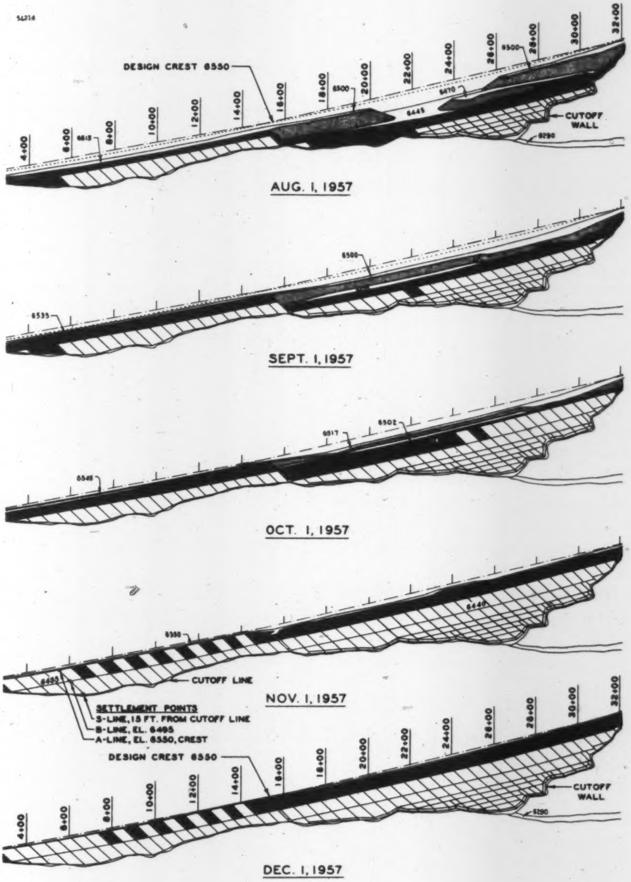

DESIGN CREST 6550
4+00
6+00
8+00
10+00
12+00
14+00
16+00
18+00
20+00
22+00
24+00
26+00
28+00
30+00
32+00
6515
6500
6470
6445
6500
CUTOFF WALL
6290
AUG. 1, 1957
6535
6500
SEPT. 1, 1957
6545
6517
6502
OCT. 1, 1957
6550
6440
6495
CUTOFF LINE
NOV. 1, 1957
SETTLEMENT POINTS
S-LINE, 15 FT. FROM CUTOFF LINE
B-LINE, EL. 6495
A-LINE, EL. 6550, CREST
DESIGN CREST 6550
CUTOFF WALL
6290
DEC. 1, 1957

FIG. 11.—MONTHLY CONSTRUCTION PROGRESS ISOMETRICS, WISHON DAM

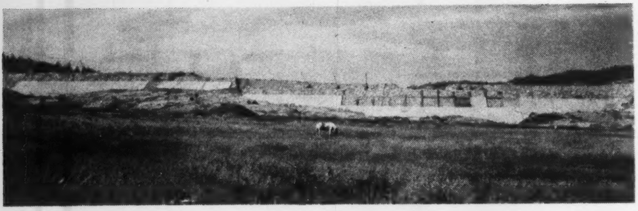


FIG. 12.—CONSTRUCTION VIEW OF JUNE 10, 1957, WISHON DAM

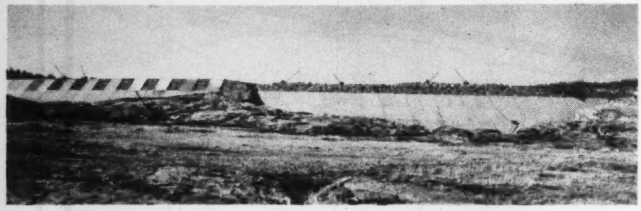


FIG. 13.—CONSTRUCTION VIEW OF NOV. 1, 1957, WISHON DAM

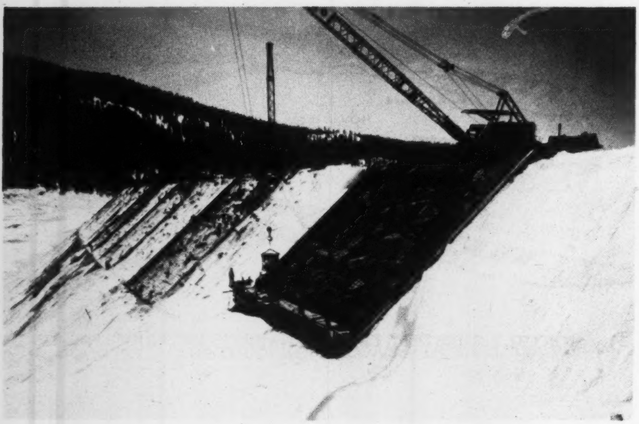


FIG. 14.—COMPLETION IN WINTER WEATHER, WISHON DAM

were observed. Due to the deeply jointed rock at Wishon many additional holes were necessary, and 26 of 300 grout holes took 100 to 700 sacks of cement.

Dumped Rockfill.—The spillway quarry and a main quarry near the right abutment, Fig. 7, furnished the rock for the dam. References (3), (6), (7) and (9) give a thorough description of quarrying, hauling and sluicing the rockfill. The powder factor was high, 1.0 lb. per cu. yd., due to deep fissures in the rock. Payment was made at a specified 50 cents a solid cubic yard for all material removed from the quarry site, including overburden. A bid price was paid for the yardage in the pay lines of the dam. The bid price included haul roads, hauling, sluicing, scarifying of lift surfaces and the additional cost of quarrying over the fixed payment. The fixed price is intentionally less than quarry cost. It gives both owner and contractor an interest in having minimum waste and reduces risk to the contractor if overburden or waste is excessive. Upon completion of the dam, it was computed that the dumped rockfill contains 32% voids. The rockfill at Courtright at the end of 1957 also was computed to contain 32% voids.

Placed Rock.—The placed rock production rate was greater than on previous dams of this type, averaging 135 and peaking at about 200 cu. yds. per 9-hour, 7-man crane shift. The reasons are: (a) simplified requirements of the specification; (b) the convenient work access for the operation of cranes and supplying of rock that is provided by the face lifts; (c) the use of very large, 6 to 15 ton, rocks; and (d) the well planned supply system and general organization of the contractor, Morrison-Knudsen Company. The large placed rocks were handled by a Lewis pin in a drilled hole or by two or sometimes three Sullivan pins in holes drilled at angles.(6,7)

Concrete.—The 3000 psi concrete mix contained aggregate in 4 sizes: #4 to 3/8, 3/8 to 3/4, 3/4 to 1-1/2 and 1-1/2 to 2-1/2. The four rather than normal 3 sizes were used because of the friable nature of the aggregate and the importance of dense grading for maximum frost resistance. It was rescreened at the batching plant. The mix contained Permanente Type II cement, 15% Airox pozzolith, Possolith 3 (water reducing agent), and Dresinate XX (AEA) air entraining agent. A cement-pozzolan factor of 4.85 and slumps of 1 to 2 inches were used. Compressive strengths at 28 days were generally 3300 to 4000 psi with few cylinders below 3000. At 90 days tests exceeded 5000 psi. Concrete was mixed in a 1.5 cu. yd. paver on the dam and placed by crane and bucket from a face lift. Concrete slabs were poured very soon after the placed rock was placed, which is not theoretically ideal since the slabs must then take subsequent construction settlement. However, this scheduling makes convenient and minimum-cost construction possible, and the construction settlement is nominal and does not damage the slabs. Cold weather concrete was mixed with 120° to 150° water and the minimum temperature specifications were met with minimum air temperatures as low as zero Fahrenheit.(11)

Settlement of Concrete Face During Construction.—The early construction of face slabs subjects the slabs to settlement during construction. The lower construction cost with this practical scheduling is considered considerably more important than the reduction of slab settlement by delaying the pouring of face slabs. The dumping of the higher lifts causes settlement of the upper slabs that have been poured. However, the weight of the new dumped fill of the upper lifts does not have much effect on the fill underlying the slabs below the upper slab. The slabs further up in the dam and away from the more rigid abutment slabs are subjected to the most construction settlement. In the several months after a slab is poured, the top settles downward more than the bottom. In fact, the bottom sometimes moves upward. This tends to cause compressive stresses in a line up the face and a closing of the soft horizontal joints. In general, settlement during construction was nominal and no cracks occurred in the concrete face. Some of the measurements that substantiate the above general comments will be summarized briefly and separately for the wing and main sections of the dam. Reference to Fig. 11 is helpful to see the stage of construction, and the location of settlement points on the dam are shown on the November 1 isometric of Fig. 11. Main Section construction settlement is shown on Fig. 15. The settlement of the face slabs during construction at Wishon is greater than that at Lower Bear River No. 1(10) due to the difference in scheduling and difference in face slopes.

Wing Section.—The wing section is 1500 feet long and generally of 100 to 150 foot height. Settlement points are on the corners of each slab, except for those located 15 feet from the cutoff line. The points 15 feet from the cutoff line (S points) and on the 6495 line (B points), about the center of the face, were set on September 1. 1957. Two months later, October 30, readings were taken again. Fifteen feet of fill and placed rock, and some concrete slabs, had been added, Fig. 11. The points 15 feet from the cutoff line moved downward vertically from zero to 0.04 feet. The B points moved "upward" from 0.00 to 0.06 feet and "upstream" 0.00 to 0.02 feet. Upward movement does actually occur. A possible explanation is that as the top fill is added, there is an upstream as well as downward movement of the rock underlying the upstream face, and since the face can't move upstream and downward without high compressive stresses, it resists and moves slightly upstream and upward. On May 13 of 1958, 6.5 months later and just before filling the reservoir, the S-points from Station 6 + 00 to 9 + 00 had moved downward 0.04 to 0.05 feet. During the same 6.5 months the B-points settled downward some 0.02 to 0.06 feet. All the movements during construction were very small and caused no cracks.

Main Section.—Some data on settlement of the concrete face of the main section during construction is presented in Fig. 15. During the two months of September and October 1957, Fig. 11, a 50 to 80-foot height of rockfill was added to the dam after the slabs between the C and D-lines had been poured. At the sections of maximum settlement, Station 21 + 60, the C point settled 0.68 feet while the D point settled 0.13 feet, Fig. 15. All settlement along the C line was downward during this two months whereas the ends of the D line moved "upward". The same characteristics may be observed for the B and C lines between the readings of October 30, 1957 and May 13, 1958.

Wishon Crest Settlement

Wishon dam was filled rapidly between May 9 and May 27. Four readings, three increments, of vertical crest settlement are plotted on a profile of the dam along the axis, Fig. 15. There is missing data at both ends of the dam because of limited time available to make a complete check of the survey data, at the time of completing this paper.

December 13, 1957 to April 21, 1958, Fig. 15.—All rockfill had been completed by December 13, 1957, Fig. 11, and the dam was being snowed in. Only a few top slabs had been poured so temporary crest settlement points were

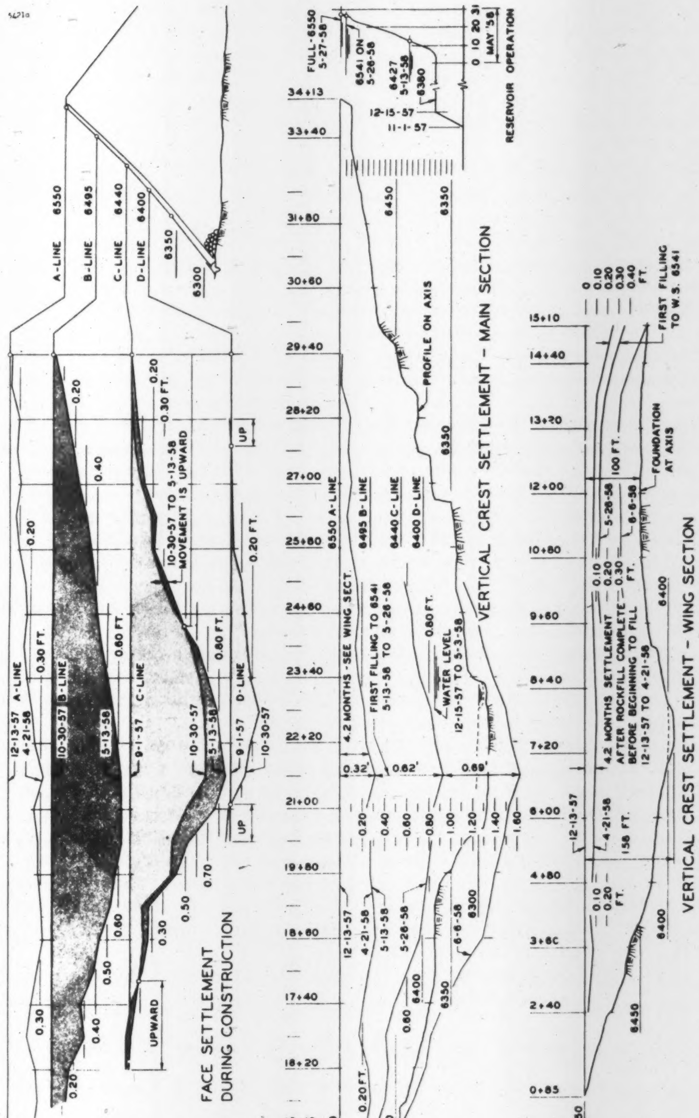


FIG. 15.—FACE SETTLEMENT DURING CONSTRUCTION, 1ST FILLING CREST SETTLEMENT, WISHON DAM

set on the crest to record movement due to aging of the dam during the winter. On April 21, 1958, 4.2 months later and just before construction of the remaining 40 top slabs, the crest had been cleared of snow and measurements were taken. During the 4.2 months the water level was at 6380, which gave a 100 foot depth of water on the maximum section. This partial filling was to keep the intake grizzly free from ice and it should have had little effect on the crest settlement. For the 4.2 months, maximum vertical settlement on the wing section was 0.10 feet (158 ft. ht. at axis) and on the main section 0.32 feet (260 ft. ht. at axis).

May 13, 1958 to May 26, 1958, Fig. 15.—During these 13 days the reservoir filled to 9 feet from the crest, from water surface 6427 to 6541. Crest settlement between the above dates was 0.62 feet at the maximum section.

May 26, 1958 to June 6, 1958, Fig. 15.—The top 9 feet of the reservoir filled during the 24 hours after the May 26, 1958 measurements. The settlement measured for this interval represents that due to the application of the last 9 feet of head and the first ten days of full reservoir pressure, and at maximum section was an additional 0.69 feet.

Settlement Due to First Complete Filling.—The vertical crest settlement of the maximum section, due to the first complete filling plus 10 days of full reservoir, is 1.31 feet or 0.5% of the 260-foot height at axis.

The main section was constructed from right to left abutment, which gives maximum settlement, during dumping, to the fill on the right abutment. The crest measurements indicate this to be true. The 170-foot high Station 18 + 60 settled more than the 210-foot high Station 25 + 20. The construction settlement readings of the B and C-lines also show the same characteristic, Fig. 15.

There is a cliff in the foundation of the dam near Station 20 + 40, Fig. 15 and 1. The effect of the cliff on settlement is reflected in the C-line readings, but not on the readings for the higher A and B-lines. The abrupt foundation contours do not appear to cause abrupt changes in the higher portion of the face, which is favorable. The vertical settlement along the crest is a considerably smoother curve than the foundation profile under the crest.

Opening and Closing of Joints at Crest.—Measurements across the joints on the crest have been made every day (to 0.001 feet readings) since beginning of filling, to detect any sudden movement if it were to occur and to determine when the joints move. No joint on the wing section, to Station 14 + 40, changed as much as 0.005 feet. On the main section crest joint openings of greater than 0.01 feet were: Station 14 + 90, 0.03 feet; 15 + 10, 0.03; 0.015; 17 + 40, 0.025; 18 + 60, 0.015; 29 + 40, 0.04; and 30 + 00, 0.045. Between Station 20 + 40 and 25 + 80, all joints closed slightly from 0.005 to 0.02 feet. These joints were constructed 2 inch open to allow for closing, VERTICAL JOINT DETAIL of Fig. 2. Joints 26 + 40 to 28 + 80 did not change. The joint openings took place between May 20 and 24 while the reservoir level raised from 6490 to 6530. As and after the reservoir completed filling the opened joints began to close gradually, as much as 0.01 feet. The closing of joints took place later and more gradually than the opening, and all movement was in one direction. The joint movements are as would be predicted, and are favorably small.

The timing and nature of joint movements are consistent with previous measurements. The magnitude of the movement is on the order of the small movements at Lower Bear River Dams.(10) The opening of joints near the top of the dam and near the abutments occurs just before complete filling, when the major settlement normal to the face in the lower portion of the dam is taking place. As the reservoir completely fills and the crest completes its initial settlement, the crest length shortens. This tends to close all joints, including those which previously opened.

Data and discussion has been confined to a brief presentation of the recently acquired crest settlement data. There are bronze pins in the corners of each slab. The measurements determine the locations of the pins in space, joint opening and closing, and change in length of the slabs. The change in length of the slabs gives a measure of the stresses in the slabs, and an indication of the movement of the rockfill under the slab. Points are also located on the downstream dumped rock face.

Courtright Dam

The site for Courtright Dam is a narrow granite canyon. Upstream is a very favorable reservoir site for a Sierra reservoir at such a high elevation, 8188 feet.(2) The dam is nine miles by road from Wishon Dam, and is located on Helms Creek, a tributary to the North Fork of the Kings.(2,8) Water released from Courtright goes to Wishon Reservoir. Annual carryover storage normally will be stored at Courtright, but the reservoir will be nearly or completely drained following a dry year.

The Courtright damsite had a narrow boulder filled gorge some 50 feet deep, Fig. 16. The left abutment was massive granite with "onion skin" surface, Fig. 20 and 21, and with talus of small rocks and fines. The right abutment was of blocky granite with jointing planes and had a talus containing many large rocks, Fig. 18. The choice of type of dam narrowed down to concrete arch and impervious face rockfill. Being a good arch site automatically eliminated a concrete gravity type. There being no impervious materials available eliminated consideration of a core rockfill type. The concrete face rockfill dam was adopted on the basis of estimated cost being 35% lower than that of the concrete arch dam. Estimates were rather reliable since Wishon rockfill dam and the Monticello and Donnells arch dams had only recently been bid. The Courtright profile is very similar to Monticello and Donnells profiles.

The design of Courtright is identical in many respects to that of Wishon and will be discussed only as it differs from Wishon. Comparative general data is presented in Table I. The data on Courtright Dam in Table I is different from previously published data. Between the 1957 and 1958 construction seasons it was decided to increase storage from 102,500 acre feet to 123,300 acre feet at the Courtright site.

Courtright Design

Diversion Tunnel and Outlet Works.—The diversion tunnel is located in the massive granite of the left abutment in preference to a shorter tunnel in the jointed rock of the right abutment, Fig. 16. The alignment passes under the end of the dam in order to permit the installation of a Stevens water surface detector in a 4 inch drill hole from crest to tunnel. The submerged tower is similar to that at Wishon.(8) An underground valve chamber, similar to that of Wishon,(8) houses a 72 inch butterfly valve and a 42 inch Howell-Bunger valve. The tunnel is unlined for the full length. The sizes, Fig. 16, were determined by permanent outlet requirements and not temporary diversion requirements, since the drainage area is small and the design permits relatively

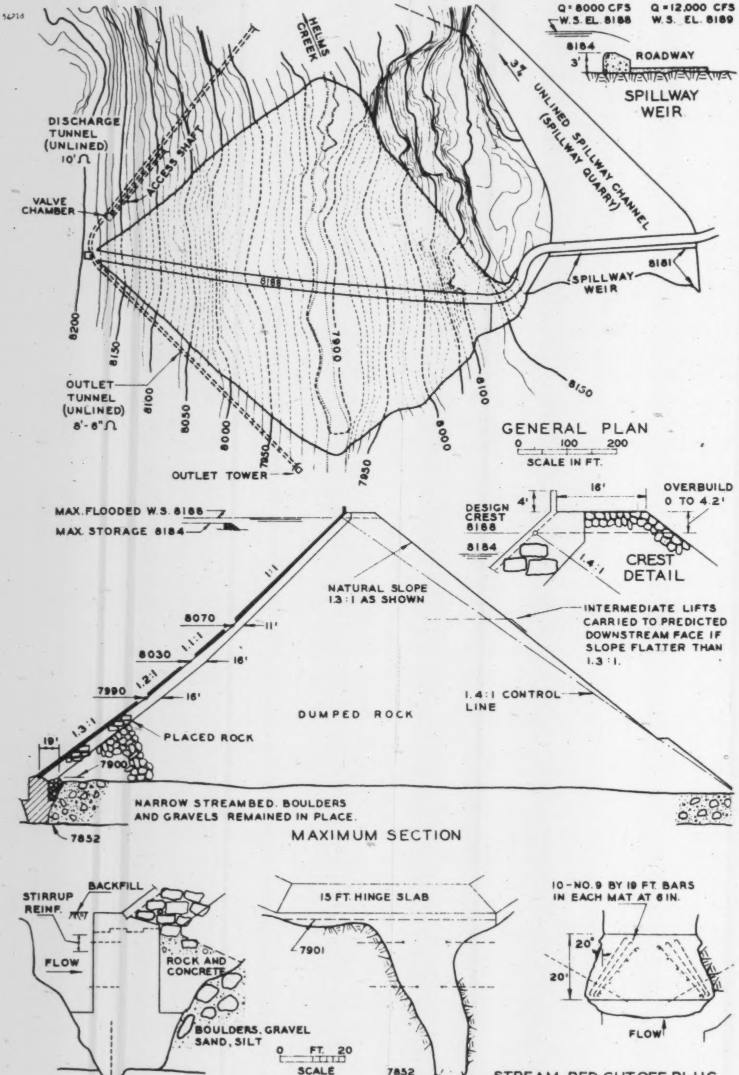

HELMS CREEK
DISCHARGE TUNNEL (UNLINED) 10' ∩
ACCESS SHAFT
VALVE CHAMBER
OUTLET TUNNEL (UNLINED) 8'-6" ∩
OUTLET TOWER
UNLINED SPILLWAY CHANNEL (SPILLWAY QUARRY)
3%
SPILLWAY WEIR
8181
8188
8200
8150
8100
8050
8000
7950
7900
GENERAL PLAN
0 100 200
SCALE IN FT.
Q = 8000 CFS W.S. EL. 8188
Q = 12,000 CFS W.S. EL. 8189
8184
3'
ROADWAY
SPILLWAY WEIR
MAX. FLOODED W.S. 8188
MAX. STORAGE 8184
1:1
8070
11'
1.1:1
8030
16'
1.2:1
7990
16'
1.3:1
19'
7900
PLACED ROCK
DUMPED ROCK
NATURAL SLOPE 1.3:1 AS SHOWN
INTERMEDIATE LIFTS CARRIED TO PREDICTED DOWNSTREAM FACE IF SLOPE FLATTER THAN 1.3:1.
1.4:1 CONTROL LINE
NARROW STREAMBED. BOULDERS AND GRAVELS REMAINED IN PLACE.
7852
MAXIMUM SECTION
DESIGN CREST 8188
4'
16'
OVERBUILD 0 TO 4.2'
8184
1.4:1
CREST DETAIL
STIRRUP REINF.
BACKFILL
FLOW
ROCK AND CONCRETE
BOULDERS, GRAVEL SAND, SILT
15 FT. HINGE SLAB
7901
0 FT. 20
SCALE
7852
10-NO. 9 BY 19 FT. BARS IN EACH MAT AT 6 IN.
20°
20'
FLOW
STREAM BED CUTOFF PLUG

FIG. 16.—PLAN, SECTION, CUTOFF PLUG, COURTRIGHT DAM

small flood flows to pass through the partially completed rockfill. The valve chamber also contains a 2 foot diameter bypass with two gate valves for draining the reservoir in the unlikely event of trouble with the main outlet.

Spillway.—The right abutment quarry provides a long open crest spillway that will pass an 8000 cfs design flood with only 4 feet of depth and with the 4 foot coping as freeboard, Fig. 16. The unlined discharge channel is designed to carry increased flow as the reservoir water surface encroaches upon the freeboard of the coping wall. This gives additional security at no cost. The road is located in the spillway channel since reservoir operation is such that spilling nearly always may be prevented if necessary. An ungated spillway is economic at Courtright, whereas it was not at Wishon because of the smaller spillway capacity and the lower incremental cost of a foot of height of dam at the Courtright site.

Cross Section.—The 1.4:1 control line for the downstream face slope intersects the upstream face slope at the design crest elevation, Fig. 16. This means that nearly the full height of the downstream slope will be at the natural dumped slope. The crest width of 16 feet is considered a practicable minimum for construction and for such a high dam. The 20 foot crests at Lower Bear River and Wishon were to provide two-way roads and permit passage over the crest when maintenance equipment was parked on the crest. These requirements were not imposed at Courtright.

Alignment.—The design axis of the dam is along a segment of a 5000 foot radius curve at design crest elevation. The layout of the dam is from a chord, called the baseline, connecting the intersections of the axis and the abutments. The vertical joints are in planes perpendicular to the baseline, Fig. 17. At the crest the middle ordinate is 18 feet and at horizontal joint 8030 it is 3 feet. In the lower area of the face the predicted settlement will nearly change the moderate upstream arching to a straight line. At the crest enough curvature will always remain to give a favorable appearance. The dam is cambered with 4.2 foot overbuild at the maximum section.

Streambed Cutoff Plug.—The streambed material consisted of boulders tightly embedded in sand and gravel. It was considered a suitable foundation for a rockfill but was excavated to provide a cutoff for the dam. The streambed cutoff plug, Fig. 16, was designed as a shear plug with a shear stress of 150 psi. Near the top of the plug, stirrups were used.

Cutoff and Grouting.—The left abutment presents no cutoff or grouting problems since the granite is massive and without seams other than the "onion skin" near the surface, which was removed. The right abutment, with its deep jointing planes, requires special consideration and a careful and thorough grouting program.

Concrete Face.—The only difference from the face at Wishon is the joint arrangement, Fig. 17. The sloping joint is used for the full perimeter to accommodate the high differential movements that are anticipated due to the steep abutments. The right abutment is so steep that several sloping joints have been used. Only a short distance out on the face from the right abutment cliff, the water pressure is carried through a long distance in rockfill to the foundation. High settlement will occur close to the fixed and unyielding cutoff wall and the several hinged slabs should tend to prevent cracks. Between the slab and its seat on the cutoff, Fig. 3, a 2-inch strip of styrofoam is used for the full length of cutoff wall to permit the slab to move at the cutoff line.

The design, as it relates to foundation excavation, can best be reviewed along with comments on construction. The dumped and placed rockfill

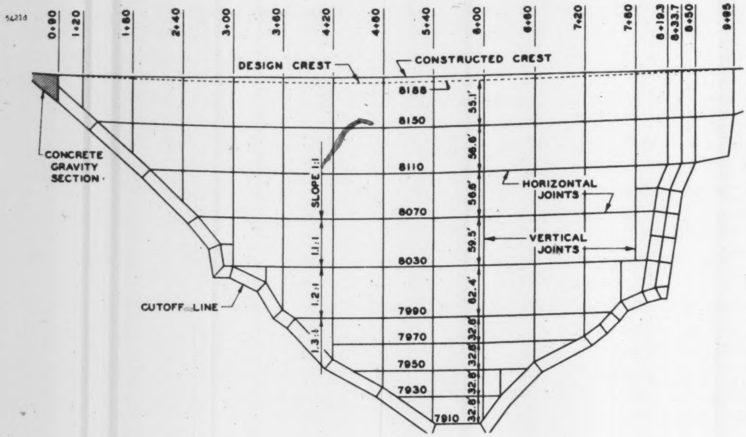

54214
0+90
1+20
1+80
2+40
3+00
3+60
4+20
4+80
5+40
6+00
6+60
7+20
7+80
8+19.3
8+33.7
8+50
8+95
DESIGN CREST
CONSTRUCTED CREST
8188
8150
8110
8070
8030
7990
7970
7950
7930
7910
55.1'
56.6'
56.6'
59.5'
62.4'
32.6
32.6
32.6
32.6
CONCRETE GRAVITY SECTION
SLOPE 1:1
1.1:1
1.2:1
1.3:1
HORIZONTAL JOINTS
VERTICAL JOINTS
CUTOFF LINE

FIG. 17.—DEVELOPED VIEW OF FACE, COURTRIGHT DAM

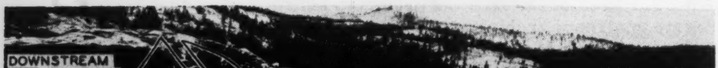

DOWNSTREAM TOE
CREST
CUTOFF
FLOW

FIG. 18.—TALUS REMAINS IN PLACE AS ROCKFILL

specifications and methods are the same except that sluicing is at a water to rock volume ratio of 2:1 rather than 3:1. This change from Wishon was made since clean rock was expected and there is not quite enough summer water supply at Courtright, for the 3:1 ratio, without recirculating.

Courtright Construction

A nine-mile road was constructed from Wishon Dam to the Courtright damsite in late 1955. The contract for the dam was awarded to Morrison-Walsh-Perini in December 1955. In four months of 1956, July through October, construction consisted of clearing, diversion tunnel, streambed plug, excavation, cutoff and opening of quarry. By asphalt paving the road in 1956, and keeping it snow-plowed during the winter, an early start in May of 1957 gave a longer than normal construction season in 1957. Six months of construction in 1957 brought the dam to the stage shown on Figs. 19B and 21. Due to very heavy snows in the winter of 1957, construction was not underway until early June of 1958, and then in six to eight feet of snow. With favorable weather at the end of the 1958 construction season, it may be expected that the dam will be completed in 1958.

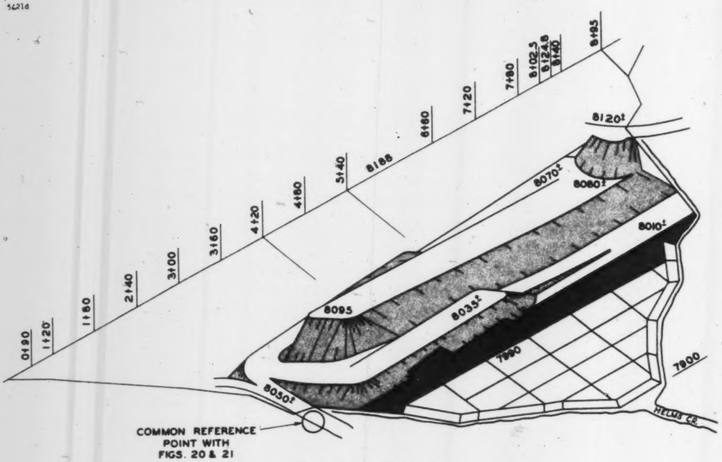
Since Wishon and Courtright are located near each other and were under construction at the same time, the prior published articles on construction cover the two dams together.(1,3-7,9)

Foundation Excavation.—Though the Courtright site is primarily exposed granite, there was a substantial amount of talus and streambed deposit. On the left abutment there was a talus of small rocks and fines that was removed. The streambed boulders and gravels were left in place except for the excavation for the streambed plug and of several sandbars. The very substantial volume of talus on the right abutment, Fig. 18, required special consideration. It was considered that this talus probably would be as good as a rockfill and the cost of removing it and replacing it with rockfill could be avoided. Five drill holes indicated the depth of talus to be 30 to 50 feet and the composition to be rocks and earth. The major portion of the talus was downstream from the axis of the dam. It was decided to leave the talus in place subject to:

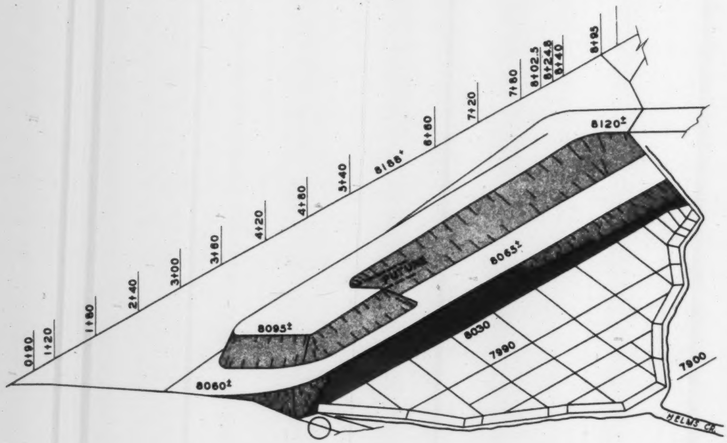
(a) Excavation of the talus to 150 feet downstream from the cutoff to assure knowledge of conditions underlying the lower half of the concrete face.
(b) To reconsider leaving the talus in place after seeing the excavation of (a).
(c) To hydraulic monitor the surface areas of fines on the talus slope at a bid price per monitor-hour.
(d) To explore and to remove some fines from the top of the talus slope when the first dumped fill lift provided access to the top of the talus.

Except for the arbitrary excavation (a) and some pockets of fines (d) the talus remained in the dam.

A suggested lift arrangement with face lifts, similar to Fig. 5, was included in the bidders' set of drawings. The contractor, Morrison-Walsh-Perini, improved on the suggested arrangement by ramping access roads to the face lifts within the rockfill to avoid the cost of excavating haul roads in the solid rock abutments. The actual arrangement is illustrated by the isometrics of Fig. 19 and photos of Figs. 20 and 21. A common point on each view is circled. Fig. 20 shows the stage of construction on August 21 when the


0+90
1+20
1+80
2+40
3+00
3+60
4+20
4+80
5+40
6+60
7+20
7+80
8+102.5
8+124.8
8+140
8+95
8188
8120±
8070±
8080±
8010±
8095
8035±
7990
8050±
7900
HELMS CR.
COMMON REFERENCE POINT WITH FIGS. 20 & 21

19 A. OCTOBER 1, 1957


0+90
1+20
1+80
2+40
3+00
3+60
4+20
4+80
5+40
6+60
7+20
7+80
8+102.5
8+124.8
8+140
8+95
8188±
8120±
FUTURE
8065±
8095±
8030
7990
8060±
7900
HELMS CR.

19 B. NOVEMBER 1, 1957
(END OF 1957 CONSTRUCTION SEASON)

FIG. 19.—ISOMETRIC VIEWS OF CONSTRUCTION, COURTRIGHT DAM

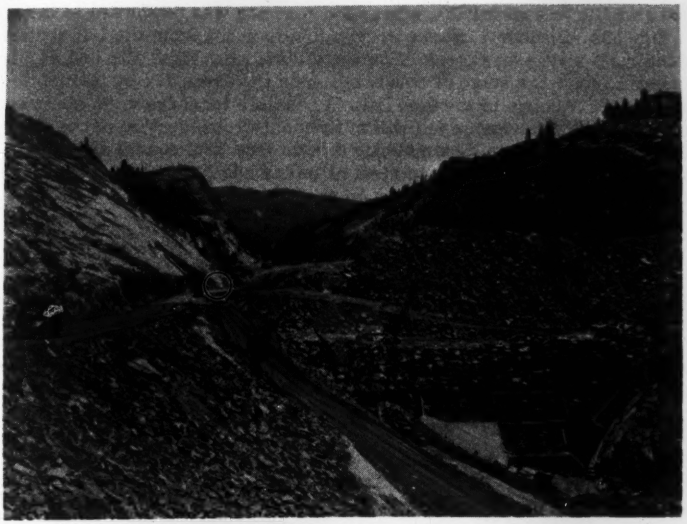


FIG. 20

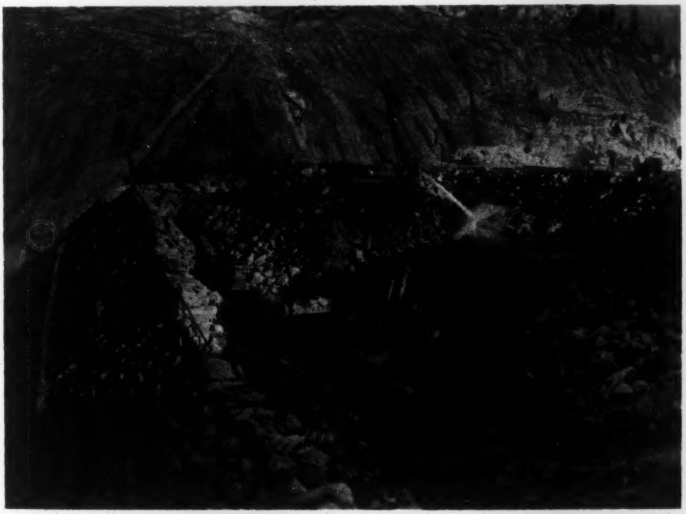


FIG. 21.—KINGS RIVER PROJECT, NEW LIFT HAS BEEN STARTED IN THE BACKGROUND IN THE CONSTRUCTION OF COURTRIGHT DAM, THE CUTOFF TRENCH CARVED INTO THE LEFT ABUTMENT BEYOND THE CRANE NEAR THE EDGE OF THE DAM IN THE BACKGROUND INDICATES THE HEIGHT STILL TO BE REACHED

dumped rockfill was rising in a spiral and ramping was downward to the face lifts. Fig. 19A, October 1, shows access to a face lift from the left abutment while the right abutment access is being established. Figs. 19B and 21, November 1, show the stage of construction on the same day by isometric and by photograph. Access to the face lifts is possible from the right abutment access road. All the ramping is within the final cross-section of the dam. The 317-foot rockfill dam is constructed from four haul roads; one near streambed level, one on each abutment at intermediate levels, and one at the crest. Two quarries were used, one on each abutment.

CONCLUSION

The design included a number of features to reduce costs over previous designs, and to make the construction of these dams a continuous and straightforward production job. Economies beyond the design changes have been made by the contractor for both dams. Their effective use of face lifts, methods in supplying and handling placed rock, use of exceptionally large placed rock, and overall job planning has kept their costs down and brought both dams well ahead of schedule.

Features were incorporated in the design and construction of the placed rock and concrete face, to effect substantial savings in cost, that knowingly subjected the face to some possible cracking. Observations, during and after construction, indicated that no cracks occurred and that settlement during construction was nominal. The concrete face and placed rock may be considered as the impervious membrane for the dam, and are treated in detail in this paper. However, the most important and main structural element of the dam is the well sluiced dumped rockfill.

Careful measurements are being kept of the movements of both dams and should contribute to future dams of greater height, still lower costs, and of unquestioned safety.

ORGANIZATION

Engineering was under the direction of Walter Dreyer, vice president and chief engineer of Pacific Gas and Electric Company, and H. V. Lutge, then chief civil engineer. I. C. Steele was consulting engineer. The experience of T. J. Corwin, supervising civil engineer, has been particularly helpful. The writer is indebted to J. E. Schumann, A. Seklemian, and A. G. Strassburger, who have been engaged with him on the engineering studies and design. The help in the review of this paper by Walter Dreyer and by J. D. Worthington, chief civil engineer, is gratefully acknowledged.

Construction is under the direction of A. J. Swank, vice president in charge of general construction, and H. W. Haberkorn, manager of hydroelectric construction. Joe Pirtz, Jr. is project superintendent and George Thacher, project engineer.

The contractor for both dams is Morrison-Walsh-Perini, sponsored by Morrison-Knudsen Company and represented by Jim Wells, vice president. B. L. Perkins is project manager and John F. Erdle is project engineer.

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DISCUSSION

G. I. DAVEY,[3] F. ASCE.—The details of design of the Wishon and Courtright Dams as they compare with the Salt Springs and Lower Bear River Dams reveal that a great deal more attention has been given to the details of construction operations so that savings can be effected by design particularly in relation to side slopes and facing rock. It seems that the thickness of placed rock has been dictated by construction reasons rather than design, in that ample width facilitated dumping methods.

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[3] Cons. Engr., Gutteridge, Haskins & Davey, Sydney, Australia.

It may be of interest to consider two dams built during 1957 in Australia, one at Mount Isa, Queensland, on the Leichhardt River and the other at Mary Kathleen on the Corella River. The dams are very similar in construction and on similar river systems, about fifty miles apart.

The dams were constructed for water supply purposes, in areas where the rainfall is seasonal and averages about fourteen inches. Evaporation is high and totals about nine feet per annum. Good run-off from the catchment areas is dependent on short period intensive rains which will give a run-off.

Details of the dimensions are set out in Table 2.

It should be noted that the height of these dams was limited by the structure of the river valleys and they could not be substantially higher.

TABLE 2

SITE	CORELLA RIVER	LEICHHARD RIVER
Catchment Area	140 sq. miles	447 sq. miles
Storage	10,600 ac.ft.	61,000 ac. ft.
Estimated maximum flood discharge	30,000 cusecs	60,000 cusecs
Nature of rock foundation	Silicified limestone	Quartzite
Spillway construction	Side cut with automatic gates	Side cut with concrete sill
Height at axis	75'	87'
Length of crest	480'	850'
Side Slopes		
U.S.	1:1¼	1:1¼
D.S.	1:1¼	1:1¼
Facing		
Placed rock	2'	2'6"
Facing		
"Gunite"	4" to 3"	5" to 3"

Leichhardt River Dam

The dam is sited on solid rock where the Leichhardt River breaks through a quartzite ridge. The body of quartzite is flanked by silicified shales.

No diversion works were provided. The risk of flooding by out of season rainfall was covered by insurance.

A spillway was excavated on the northern bank from which some 100,000 cubic yards of rock were obtained for dam construction and 100,000 cubic yards dumped to waste. The balance of fill required for the dam was quarried from the quartzite masses adjacent to the dam site. The site was closely drilled and grouted to a depth of 30' and very little grout was injected.

Construction was carried out by dumping from 12 yard trucks loaded by 2-1/2 yd. shovels in the quarries. The fills were limited to a maximum of 30' and generally lesser heights were used. Dumping was controlled to ensure maintenance of the designed slope lines. The fill was constantly trimmed and traversed by a heavy bulldozer. The face of each dumping point was kept wet by constant hosing using 1" nozzle at 100 p.s.i. Each truck delivering rock to the fill paused en route under a 4" diameter nozzle so that the contents of the truck were wetted and the voids in the truck body filled with water. When dumped the wet rocks with water slid down the wetted face of the fill, tended to drag the fill down and maintained velocity in falling to the bottom.

This method of watering was adopted in the first case because of the difficulty of obtaining sufficient water. The Leichhardt River is dry almost the whole year and the only water available was in a small water hole immediately downstream of the dam site. The water pumped from this hole and used in facilitating consolidation was drained back into the water hole and recirculated. The method of filling trucks with water was continued when the excellent consolidation results obtained were noted.

There has been no measurable settlement in the fill since it was completed. Certainly the dam has not yet filled because of low rainfall and run-off though it has some forty feet of water.

The calculated voids in the dam amount to 30% of the total volume.

The dam was faced by selecting stones from the fill and packing in place by hand for a thickness of about 2'. This surface was then covered with a lean (10:1) sand-cement mixture using a cement gun. On this surface formwork and scaffolds were set up and panels of reinforcing mesh placed. The forming held dumb-bell type rubber jointing strips surrounding each panel, which was approximately 30' square. The surface sand-cement mixture (3:1) was then placed by cement gun, the thickness varying from 6" to 4". After placing the surface was sprayed with curing wax covered with sheets of hessian and kept damp for three weeks.

Some fin cracks have appeared in the surfacing where exposed to the air but they are unlikely to allow of leakage.

The dam has performed very satisfactorily to date and there is no evidence of leakage.

Corella River Dam

The site is in solid rock, and was drilled and grouted. Very little grout could be injected to a depth of 30'. (It has been found by subsequent drilling that isolated cavities occurred in the silicified limestone and additional grouting has been carried out).

Again, no diversion works were constructed as the river bed is dry except in times of rain, and it was proposed to construct the dam in the period between June and November when rain rarely falls.

Again, a spillway was excavated, on the southern bank and the whole of the material required for the construction of the dam was obtained from the cut. The total excavation was 100,000 cubic yards and the fill in the dam was 110,000 cubic yards.

The fill was made by dumping from 12 yd. trucks loaded by a 3 C. yd. shovel. The maximum depth of fill at any one time was 30' but generally 20' was used. The dumping was done with care to maintain the designed slopes, and the procedure was exactly as described for the Leichhardt River dam except that the rock was sluiced as it was dumped from the truck and on the dump using two 1-1/2" nozzles. Generally the quantity of water was equal to twice the rock volume. Water for sluicing was obtained from a large water hole immediately upstream of the dam site. The trucks were not filled with water, as at Leichhardt River Dam, until the fill was almost finished, when the idea of filling the truck was developed.

The voids in the fill were calculated at 32% of the total volume.

KARL TERZAGHI,[4] Hon. M. ASCE.—Mr. Cooke's paper on the Wishon and Courtright Dams is an outstanding contribution to the symposium because it contains a vast amount of specific and reliable information concerning expertly designed and constructed rockfill dams. The following discussion will be limited to the topic illustrated by Fig. 3 of the paper. The figure illustrates three stages in the development of the design of the cutoff for rockfill dams on a rock foundation. The development took place during a period of about 25 yr.

In order to establish a rational basis for comparing the relative merits of the three types of design shown in the aforementioned Fig. 3, the functions of the cutoff must be considered. These are illustrated by the writer's Fig. 22 representing an idealized vertical section through a jointed rock and a concrete cutoff. The rock is assumed to contain three sets of joints intersecting each other at right angles. In each set the spacing of the joints is D and the width of the joints is t.

If the concrete cutoff is not supplemented by a grout curtain the water would enter the joint system at a and discharge into the base of the rockfill at b. At b the roof of each horizontal joint is acted upon by the seepage pressure of the percolating water which tends to lift the blocks located between the vertical joints off their seat. This danger can be eliminated completely by covering the exit area b with a graded transition layer. If the size of the particles resting directly on the rock surface is equal to or smaller than the size of the blocks between joints, the blocks between joints cannot rise. Therefore the cutoff serves only the purpose of reducing the seepage losses to a tolerable value. These losses can be estimated by the following reasoning.

According to the laws of the laminar flow of water through a slit-shaped conduit with a width b (in cm) the quantity q of water which flows through the conduit per unit of breadth is

$$q = i\frac{w\ b^3}{12\ m_v} \quad \ldots\ldots\ldots\ldots (1)$$

wherein i is the hydraulic gradient, w denotes the unit weight of water and m_v = coefficient of viscosity of the water.

The water percolates through the joint system shown in Fig. 22 through two sets of joints at right angles to the third one. The rate of seepage q_A through a portion A of a section parallel to this third set is equal to the rate of seepage q per unit of breadth of the joints crossing the area multiplied by the total length N_A of the lines of intersection between area A and the joints. If area A has the shape of a square $B \times B$, $N_A = 2\,B^2/D$. Therefore the seepage through area A is

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[4] Prof. Civ. Engrg., Div. of Engrg. Sciences, Harvard Univ., Cambridge, Mass.

$$q_A = q\,\frac{2\,B^2}{D} = \frac{w\,b^3}{12\,m_v} \times \frac{2}{D} \times i \times B^2 = k\,i\,B^2 \quad \ldots\ldots\ldots\ldots (2)$$

The coefficient

$$k = \frac{w\,b^3}{12\,m_v} \times \frac{2}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

represents the coefficient of permeability of the jointed rock. Substituting

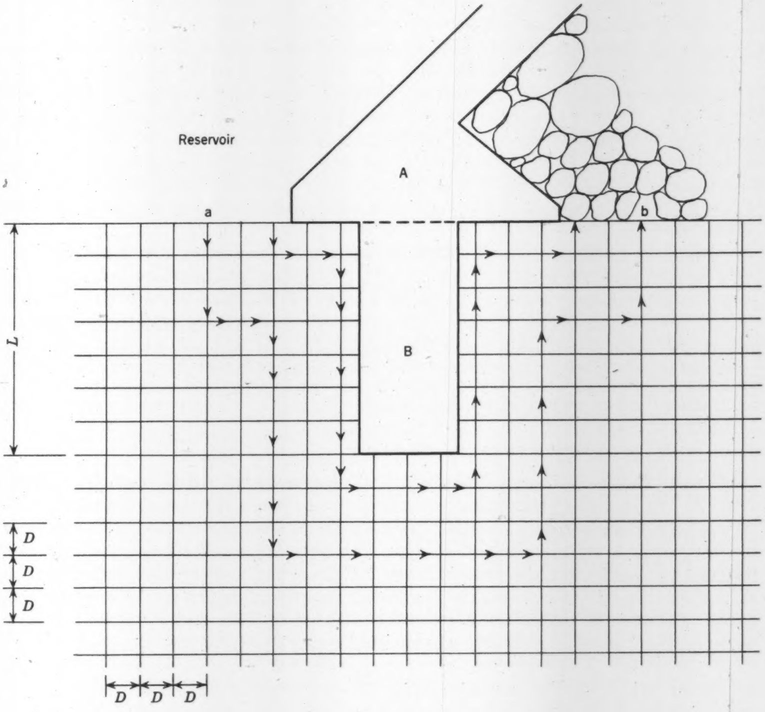

FIG. 22

$w = 1$ gm/cm^3 and $m_v = 1.15 \times 10^{-5}$ gm sec per cm^2 (viscosity of water at 60° F),

$$k \text{ (cm per sec)} = 1.44 \times 10^4 \times \frac{b}{D} \times b^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The coefficient of permeability k (cm per sec) of a sand with an effective grainsize d (cm) is roughly equal to $k = 80\,d^2$. Therefore the permeability of a jointed rock is approximately equal to that of a sand with effective grainsize

$$d\ (\mathrm{cm}) = 13.4\ t \sqrt{\frac{t}{D}} \quad \dots\dots\dots\dots\dots\dots (5)$$

If the joints in a rock are t = 0.01 cm wide and are spaced at distances of 10 cm (=D), the rock has the permeability of a sand with an effective grainsize d = 0.004 cm which is that of a fine, silty sand. If k is known the loss of water by seepage through a jointed rock around a cutoff, unsupplemented by a grout curtain, can be computed as if the cutoff has been established in a sand with the same coefficient of permeability.

An accurate evaluation of k, Eq. 4, is impracticable, but also unnecessary. The spacing D contained in the equation can be assumed equal to the average length of the intact core sections recovered from diamond drillholes. Information concerning the width t of the joints is obtained from visual inspection of rock outcrops. Noticeable water loss from drill holes indicates the presence of joints which are so wide that they can reliably be grouted.

Beneath valleys cut into bedrock the average spacing D between joints commonly increases and their width t decreases with increasing depth below the surface. This is the result of the stress relaxation associated with the removal, by erosion, of the rock which formerly occupied the valley. Below a depth of 100 ft. to 200 ft most of the joints are likely to be closed. However within a depth of 10 or 20 ft both D and t can safely be assumed to be independent of depth, unless the boring records indicate a conspicuous derivation from this rule.

Returning to the cutoff, it has been shown at the outset that the sole purpose of a cutoff along the lower edge of the upstream face of a rockfill dam consists in reducing the loss of water due to percolation through the joints of the rock to a tolerable value. In order to accomplish it at a minimum expense, the rock conditions must be considered. In this connection distinction must be made between three principal categories of rock:

a) Sound rock, joints wider than 0.1 mm,
b) Sound rock, joints narrower than 0.1 mm, and
c) Sound rock with weathered top layer.

If the joints are wider than 0.1 mm (case a) the rock can reliably be grouted. In this case the most economical procedure for intercepting the flow of seepage would be to eliminate the concrete cutoff, B in Fig. 22, to establish the foot wall, A on the cleaned surface of the sound rock and to grout the rock beneath the wall. In order to increase the pressure under which the grout can safely be forced into the joints the wall could be anchored to its base by rock bolts. Blasting a cutoff trench and backfilling it with concrete would be far more expensive and the blasting operations may even damage the rock.

If the joints in sound rock are narrower than 0.1 mm cement grout does not enter them. In this case a cutoff wall to depth L, as shown in Fig. 22, reduces the loss of water by increasing the length of the path of percolation by 2 L. However the theory of seepage through permeable materials shows that a cutoff with a depth L has no more effect on the loss of water than a blanket with a width 2L. Hence the beneficial effects of the cutoff can also be obtained, at a lesser expense, by covering the rock surface on the upstream side of the foot wall A, to a distance 2 L from the wall with an impervious blanket. In order to plug those among the joints at depths which are wider than 0.1 mm the rock beneath the wall A, Fig. 22, should be grouted.

If, or wherever, the uppermost layer of the rock is weathered, a cutoff trench should be excavated to the depth to which it can be done without blasting and backfilled with concrete. Below this depth the rock should be grouted. Experience has shown that the width of the joints is greatest at the boundary between the sound and the weathered rock. Therefore grouting can be expected to be effective.

Considering the facts set forth in the preceding paragraphs it is rather difficult to understand how the brutal practice of blasting a cutoff trench out of sound rock came into existence. It may be a vestige of the days when the technique of rock grouting was still unknown.

J. BARRY COOKE,[5] F. ASCE.—Mr. G. I. Davey has presented data on two economical gunite face rockfill dams. Mr. Terzaghi has given a rational discussion of the functions of the cutoff of a concrete face rockfill dam and has suggested further economy. The paper and the two discussions emphasize the particular importance and possibility of practical and economic construction in this type of dam.

The recently constructed 75-ft high Corella River Dam and the 87-ft high Leichard River Dam, described by Mr. Davey, are particularly economical. The 1.25 to 1 (horizontal to vertical) slopes, 2 ft or 2 ½ ft thick placed rock and thin gunite membranes are practicable and certainly very safe for this height of dam. The Upper Bear River Dam and Meadow Lake Dam, described[6] in this symposium, with steeper slopes (0.5 to 1 and 0.75 to 1), and less competent rockfill are both in good health as they approach 60 yr of age. Though the very steep slopes are neither economical nor recommended today, the experience testifies to the substantial safety of the more conservatively sloped dams of today's design and construction standards.

The sluicing by 1-in. nozzles plus watering the loads in the truck beds, though suitable for the 30-ft lifts and for the low dams, would not be considered by the author to be suitable for higher lifts on higher dams. The excavation of pockets of spalls that accumulate at the top of high lifts is considered to require jets of 2-1/2 to 3 in. in diameter. It is interesting to note that with care in dumping, the 1.25 to 1 slope was maintained in the 20-ft to 30-ft high lifts. Courtright Dam slopes were equally steep where the low topping lifts were used in the top 40 ft of the dam. The slopes at 60 ft stations were, in relation to 1 vertical: 1.25, 1.27, 1.25, 1.15, 1.15, 1.11, 1.19, 1.23, 1.12, 1.21, and 1.19. On the slope 40 to 80 ft below the crest in elevation, the slopes were: 1.37, 1.36, 1.37, 1.17, 1.27, 1.20, 1.34, 1.40, and 1.25.

Mr. Terzaghi has carefully reviewed the functioning of the cutoff for the impervious face rockfill type of dam. His conclusions, with which the author is in agreement, are: 1) the function of the cutoff is to reduce seepage or leakage loss and does not involve safety of this type of dam; 2) cutoff trench excavation is very expensive and, even when done with care, can fracture and loosen adjacent rock; 3) grouting provides an effective and economical cutoff curtain; and 4) a suitable and economical cutoff can be obtained by grouting under a rock-anchored slab.

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[5] Super. Civ. Engr., Pacific Gas and Elec. Co., San Francisco, Calif.

[6] "Rockfill Dams: Salt Springs and Lower Bear River Concrete Face Dams," by I. C. Steele and J. B. Cooke, Transactions, Vol. 125, Part II, 1960.

Mr. Terzaghi mentions a transition layer downstream from the cutoff which would prevent foundation rock from rising. The placed rock with chinked voids serves this purpose at the contact of placed rock and foundation. Conditions are such that the surface foundation rock downstream from the cutoff should not rise to increase seepage and could not rise to cause a blowout under the cutoff. To a substantial distance downstream from the axis the downward pressure of the self-draining rockfill on the foundation exceeds 100% uplift.

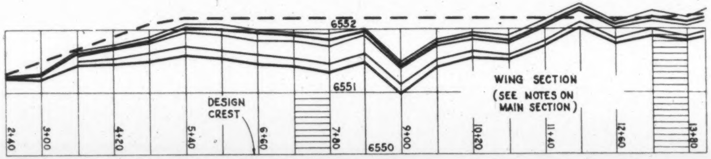
The trend to shallower cutoff walls for Pacific Gas and Electric Company dams has been due to the high cost and the feeling that depth does little to decrease seepage and improve safety. To excavate a narrow trench requires drill holes at 9-in. to 12-in. spacing, about 40 ft of drill hole per cubic yard of excavation and expensive handling of muck. In jointed rock the loosening of the rock in the walls requires going to greater depth to meet a design requirement of a specified depth with competent walls. An excavated and concrete backfilled cutoff is very expensive compared to a grout curtain.

Mr. Terzaghi emphasizes the effectiveness of grouting. In the cutoffs at the company's recent dams, grout pipes have been angled in the cutoff pour to make the drill holes cross specific seams or bedding planes in addition to pattern grouting. A grout curtain under a rock-anchored slab should provide a safe, effective and economical cutoff.

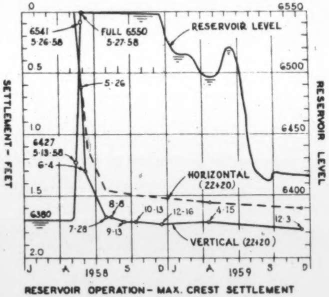
WISHON DAM

Vertical crest settlement for the first 2 yr after completion of the rockfill is shown in detail on Fig. 23. The reservoir operation is also shown. The dashed line shows the crest profile as designed and the line dated 12-13-57 (December 13, 1957) shows the position of the crest on the day dumped rockfill was completed. The actual crest was generally at only one or two-tenths of a foot lower than design elevation upon completion of the rockfill. Between December 13, 1957 and April 13, 1958, while the fill aged for the first 4 months with, essentially, an empty reservoir, the movement was small (only 0.10 ft at the 150-ft high Station 7+80 and 0.32 ft at the 249-ft high Station 21+60). The major portion of settlement took place between 5-13-58 (May 13) and 6-4-58 (June 4), which period included the 13 days of rapid filling and the first 9 days of full reservoir. The next reading was on 7-28-58 (July 28), 54 days later and about 2 months after the reservoir was full. The 7-28-58 reading is the last one plotted in the form of a profile because the settlement has been so small since that time. The tabulation shows six readings between 7-28-58 and 12-3-59 (December 3), which record very small movements during 1 yr and 4 months when the reservoir operation was as shown on Fig. 23. Between 12-16-58 (Dec. 16) and 4-15-59 (April 15), while the reservoir level went down, the crest "raised" 0.01 ft to 0.02 ft and at only 4 of the 51 crest settlement points did the points go downward slightly.

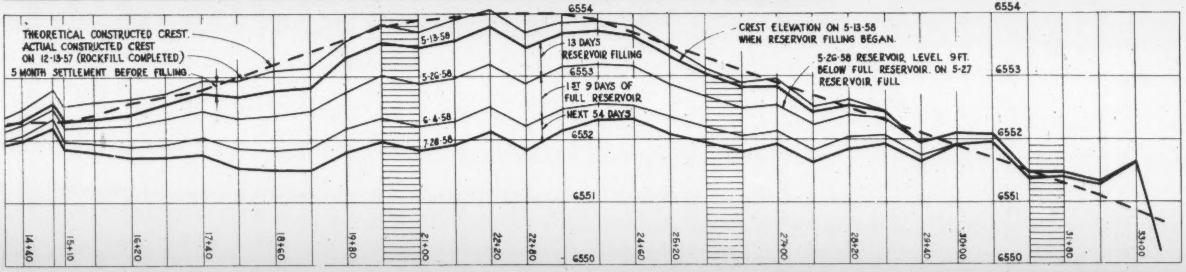
The tabulation on Fig. 23, in addition to all of the crest readings of vertical settlement, shows the height at each section and the settlement, in percentage of height, for each section. The percentage of height figure is based on the 12-3-59 (Dec. 3) reading which is 1 yr and 5 months after the reservoir was first filled. The settlement at the maximum section of the wing section, Station 7+80, is 0.39% of the height or 0.58 ft. The settlement at the maximum section of the main section, Station 22+20 is 0.69% of the height, or 1.77 ft. This is consistent with measurements of settlement on other well constructed dumped



HEIGHT AT AXIS-FT.		45	70	83	108	123	133	130	128	138	150	138	136	116	110	106	98	98	98	98	98
12-13-57 TO 4-13-58		.01	.01	.06	.04	.06	—	08	—	.04	.10	.04	.04	—	06	.08	—	—	05	.10	08
5-13-58		.00	.00	00	.00	.00	.00	.00	.00	.00	.00	00	.00	.00	00	.00	.00	.00	.00	.00	00
5-26-58	13 DAYS	.00	.05	.02	.02	.04	.08	.09	.14	.09	.11	.09	.06	.06	.06	.05	.06	.05	.05	.05	.07
6-4-58	9 "	.00	.05	.11	.17	.22	.29	.34	.36	.37	.40	.36	.32	.28	.26	.24	.24	.23	.21	.20	.21
7-28-58	54 "	.00	.06	.15	.23	.30	.39	.45	.49	.50	.52	.50	.45	.40	.36	.30	.33	.33	.30	.28	.28
8-9-58	12 "	.01	.07	.16	.24	.31	.41	.47	.50	.51	.53	.51	.46	.40	.37	.34	.34	.32	.30	.28	.29
9-13-58	35 "	.02	.08	.17	.25	.32	.41	.47	.50	.52	.54	.52	.47	.41	.37	.35	.35	.34	.31	.29	.31
10-13-58	30 "	.00	.08	.17	.25	.33	.43	.47	.51	.53	.54	.51	.46	.40	.37	.34	.33	.32	.30	.30	.32
12-16-58	64 "	.03	.09	.18	.27	.34	.43	.49	.52	.53	.56	.53	.48	.42	.39	.36	.35	.37	.32	.30	.31
4-15-59	121 "	.02	.08	.18	.25	.32	.41	.48	.51	.53	.55	.52	.47	.41	.37	.35	.35	.34	.32	.30	.31
12-3-59	232 DAYS	.03	.10	.20	.28	.35	.45	.51	.54	.55	.58	.54	.50	.45	.40	.38	.37	.35	.32	.31	.33
% OF HEIGHT		0.07	0.14	0.24	0.25	0.29	0.34	0.39	0.42	0.40	0.39	0.39	0.37	0.39	0.36	0.36	0.38	0.36	0.33	0.32	0.34



RESERVOIR OPERATION - MAX. CREST SETTLEMENT



103	103	110	130	155	166	176	183	176	168	214	264	254	249	256	244	HEIGHT AT AXIS-FT.	
.17	—	.24	.27	.25	.17	.21	.26	.31	.30	.27	.27	.24	.32	.24	.22	12-13-57 TO 4-13-58	
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	00	.00	.00	.00		5-13-58
.07	.10	.10	.14	.18	.25	.28	.34	.40	.37	.54	.55	.60	.61	.62	.59	13 DAYS	5-26-58
.24	.28	.31	.40	.52	.65	.72	.83	.96	.96	1.13	1.21	1.28	1.30	1.31	1.27	9 "	6-4-58
.32	.38	.41	.53	.69	.85	.97	1.12	1.25	1.30	1.48	1.59	1.66	1.67	1.68	1.63	54 "	7-28-58
.33	.39	.42	.54	.69	.87	.98	1.12	1.27	1.31	1.49	1.59	1.66	1.68	1.69	1.63	12 "	8-9-58
.34	.41	.44	.55	.70	.88	.99	1.13	1.28	1.31	1.50	1.58	1.67	1.71	1.72	1.66	35 "	9-13-58
.33	.40	.43	.54	.70	.87	.99	1.13	1.28	1.31	1.49	1.61	1.68	1.71	1.72	1.66	30 "	10-13-58
.35	.41	.45	.56	.72	.90	1.02	1.16	1.31	1.35	1.52	1.63	1.70	1.73	1.73	1.68	64 "	12-16-58
.34	.41	.44	.56	.72	.90	1.02	1.15	1.30	1.34	1.52	1.62	1.68	1.71	1.72	1.67	121 "	4-12-59
.36	.43	.47	.59	.75	.94	1.06	1.20	1.35	1.39	1.56	1.67	1.73	1.76	1.77	1.72	232 DAYS	12-8-59
0.35	0.42	0.43	0.45	0.49	0.57	0.60	0.66	0.77	0.83	0.73	0.63	0.68	0.71	0.69	0.71	% OF HEIGHT	

245	230	224	208	212	200	170	130	134	125	103	70	60	56	47	HEIGHT AT AXIS-FT.	
.17	.14	.17	.15	.09	.06	.10	.07	.09	—	—	—	—	—	—	12-13-57 TO 4-13-58	
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	00	.00	00		5-13-58
.55	.52	.47	.43	.36	.35	.30	.22	.15	.10	.03	01	.02	.00	.00	13 DAYS	5-26-58
1.21	1.14	1.11	.97	.85	.81	.72	.63	.50	.37	.20		10	.05	.06	9 "	6-4-58
1.54	1.42	1.32	1.23	1.09	1.03	.92	.81	.67	.50	.30	17	12	.09	.05	54 "	7-28-58
1.55	1.46	1.34	1.24	1.11	1.05	.93	.82	.68	.52	.32	18	.12	09	.05	12 "	8-9-58
1.57		1.36	1.26	1.14	1.07	.98	.84	.70	.54	.33	20	.15	.08	.07	35 "	9-13-58
1.57	1.48	1.35	1.26	1.13	1.07	.95	.84	.70	.53	.33	20	.14	.11	.06	30 "	10-13-58
1.57	1.48	1.36	1.27	1.13	1.07	.94	.85	.70	.53	.33	19	.14	.10	06	64 "	12-16-58
1.58	1.49	1.36	1.26	1.13	1.07	.96	.85	.70	.53	.32	.19	.14	.11	.06	121 "	4-12-59
1.63	1.53	1.40	1.31	1.18	1.12	.99	.88	.73	.56	.35	.20	.14	10	.06	232 DAYS	12-8-59
0.67	0.67	0.63	0.63	0.56	0.56	0.58	0.67	0.55	0.45	0.34	0.29	0.23	0.18	0.13	% OF HEIGHT	

FIG. 23.—VERTICAL CREST SETTLEMENT—WISHON DAM

rockfill dams. The settlement of sections other than these two highest and central sections is influenced by the steepness of the abutments and by the foundation contours and, as seen on Fig. 22, the settlement expressed in percentage of height at axis varies between 0.07% and 0.83%.

Fig. 24 shows profiles of vertical and horizontal components of crest settlement along with foundation profiles. The settlement is plotted relative to a straight line datum which shows the smoothness of the settlement compared to the irregular foundation. The irregular lines of Fig. 23 are due to the distorted scale and the plotting of actual elevations of the points.

It is of interest to see that the horizontal movements of points on the 150-ft high wing section exceed the vertical movements, whereas it is the opposite in the central portion of the 250-ft high main section. As is typical of concrete face dam movement, the ratio of horizontal component to vertical component of settlement of a crest point increases as points are taken for sections from the center to each abutment. For example, the ratio is 0.86 at Sta. 21+00 and 2.00 at Sta. 30+00. The influence on crest settlement of the very irregular foundation in the vicinity of Stations 18+00 and 21+00 is seen to be rather small and not abrupt.

Fig. 25 gives all of the vertical and horizontal movements, during the first year, of points above the 1959 minimum reservoir level of 6412 ft. Also shown on Fig. 25 are cross sections illustrating the settlement. The vertical movements at the corners of slabs are shown to indicate the moderate offsets between adjacent slabs. The offsets are seen to be 0.01 ft to 0.02 ft. At maximum height section 22+20, the first year movement normal to the face was 2.8 ft at elevation 6440 ft, a little above the mid-height of the dam. At Station 19+80, the movement is greater than at Station 18+60 due to the lower foundation profile downstream from the axis and due to 19+80 being closer to the center of the main section.

Table 3 and Table 4 give the opening and closing of vertical and horizontal joints for the wing section and main section, respectively. The movements were very small. Where the joint opening is smaller than noted in the paper, it is because the joint has since closed slightly. All joint movements are seen to be small and it is possible that the very large placed rock did not permit the joints to act as much as they might have and that the underlying fill had greater components of movement in planes parallel to the face.

Table 3 for the wing section shows slight (0.01 ft) opening of the crest A-line joints near the ends of the wing section and slight closing (0.01 ft) of the joints near the center. The B-line vertical joints generally opened 0.01 to 0.02 ft before filling the reservoir and then closed during the first years cycle of reservoir operation. The resultant movement was only 0.01 ft opening or closing as shown on the tabulation as TOTAL. The opening between 9-1-57 (Sept. 1) and the beginning of filling on 5-13-58 (May 13) was probably due to shrinkage of the slabs. The horizontal joint, B-line, was constructed with a 3/4 in. redwood board and Z copper seal. It closed by from 0.01 ft to 0.02 ft.

Table 4 for the main section shows, for the crest A-line, the usual opening of vertical joints near the ends of the dam and closing near the center. The vertical joints at the B-line generally did not move or opened slightly before filling began on 5-13-58 (May 13) and all closed (except at Station 19+20) during first years operation of the reservoir. The resultant movements of vertical joints at the B-line are all closing with greatest closures in the central part of the main section. For the C-line similar comments as for B-line apply. The B-line and C-line horizontal joints all closed moderately, 0.01 ft to 0.02

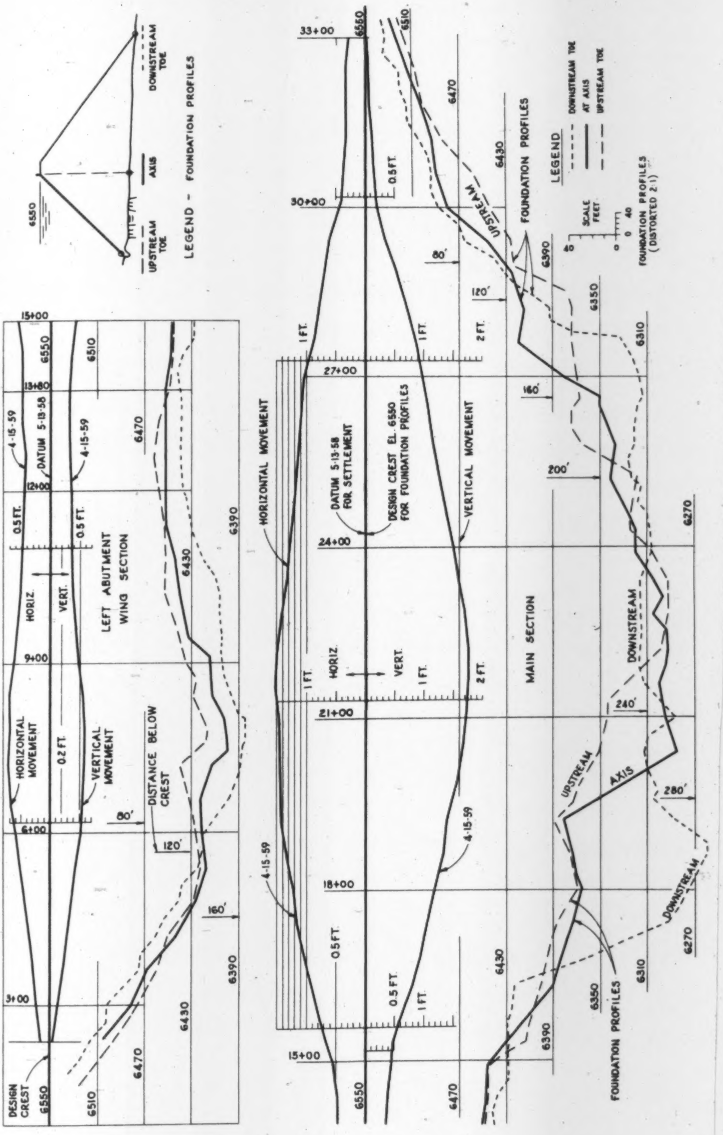


FIG. 24.—PROFILES OF HORIZONTAL AND VERTICAL CREST SETTLEMENT-FOUNDATION PROFILES—WISHON DAM

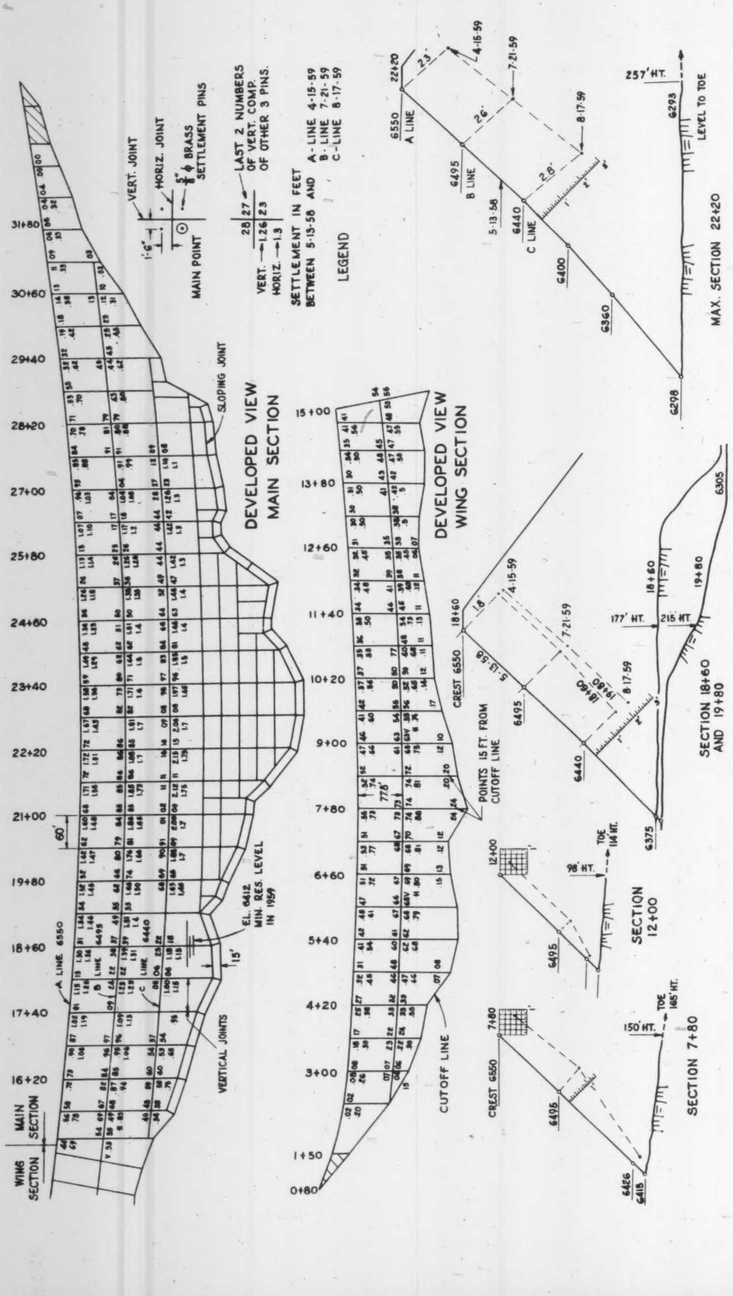

DEVELOPED VIEW
MAIN SECTION
DEVELOPED VIEW
WING SECTION
WING SECTION
MAIN SECTION
A LINE 6550
B LINE 6495
C LINE 6440
EL. 6412 MIN. RES. LEVEL IN 1959
VERTICAL JOINTS
SLOPING JOINT
CUTOFF LINE
POINTS 15 FT. FROM CUTOFF LINE
VERT. JOINT
HORIZ. JOINT
5/8" ϕ BRASS SETTLEMENT PINS
MAIN POINT
LAST 2 NUMBERS OF VERT. COMP. OF OTHER 3 PINS.
VERT. → 1.26
HORIZ. → 1.3
SETTLEMENT IN FEET BETWEEN 5-15-58 AND A-LINE 4-15-59 B-LINE 7-21-59 C-LINE 8-17-59
LEGEND
MAX. SECTION 22+20
SECTION 18+60 AND 19+80
SECTION 12+00
SECTION 7+80
CREST 6550
LEVEL TO TOE
257' HT.
177' HT.
215' HT.
98' HT.
150' HT.
TOE 114' HT.
TOE 165' HT.
4-15-59
7-21-59
8-17-59
5-13-58
16+20
17+40
18+60
19+80
21+00
22+20
23+40
24+60
25+80
27+00
28+20
29+40
30+60
31+80
0+80
1+50
3+00
4+20
5+40
6+60
7+80
9+00
10+20
11+40
12+60
13+80
15+00
6550
6495
6440
6400
6360
6298
6293
6375
6305
6426
6418
60'
15'
23'
26'
28'

FIG. 25.—FIRST YEAR VERTICAL AND HORIZONTAL SETTLEMENT DATA-WISHON DAM

TABLE 3.—VERTICAL AND HORIZONTAL JOINT MOVEMENTS—WING SECTION—WISHON DAM

	VERTICAL JOINTS - FT (+) = Opening and (-) = Closing				HORIZONTAL JOINTS - FT (+) = Opening and (-) = Closing		
	A-LINE	B-LINE			B-LINE		
	5-13-58 To 12-3-59	9-1-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL	10-30-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL
2+40	+ .01						
3+00	0		- .01	- .01	0	0	0
3+60	+ .01	+ .02	0	+ .02	- .01	- .01	- .02
4+20	0	0	- .01	- .01	- .01	0	- .01
4+80	0	- .01	0	- .01	- .01	0	- .01
5+40	- .01	0	0	0	0	- .01	- .01
6+00	- .01	- .01	0	- .01	- .02	0	- .02
6+60	0	+ .01	- .02	- .01	0	- .01	- .01
7+20	- .01	- .01	0	- .01	- .01	0	- .01
7+80	- .01	0	0	0	- .01	0	- .01
8+40	- .01	0	0	0	0	- .01	- .01
9+00	- .01	+ .02	- .02	0	0	- .01	- .01
9+60	- .01	+ .02	- .03	- .01	0	- .02	- .02
10+20	0	+ .02	- .02	0	- .01	- .01	- .02
10+80	0	0	0	0		- .02	
11+40	0	+ .02	- .02	0		- .02	
12+00	0	0	0	0	- .01	- .01	- .02
12+60	0	+ .01	0	+ .01	0	- .02	- .02
13+20	+ .01	0	0	0		- .01	- .01
13+80	0	+ .02	- .01	+ .01	- .01	0	- .01
14+40	0	+ .02	- .01	+ .01		- .01	
14+90	+ .03	0	0	0		- .01	

TABLE 4.—VERTICAL AND HORIZONTAL JOINT MOVEMENTS—MAIN SECTION—WISHON DAM

	VERTICAL JOINTS - FT (+) = Opening and (-) Closing							HORIZONTAL JOINTS - FT (+) = Opening and (-) = Closing			
	A-LINE	B-LINE			C-LINE			B-LINE	C-LINE		
	5-13-58 To 12-3-59	10-30-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL	10-30-57 To 5-13-58	5-13-58 To 8-17-59	TOTAL	5-13-58 To 7-21-59	10-30-57 To 5-13-58	5-13-58 To 8-17-59	TOTAL
15+10	+ .02		- .03	- .03				- .01			
15+60	+ .01	- .01	- .01	- .02	- .01	- .01	- .02	- .01		- .02	
16+20	0	0	- .01	- .01	0	- .01	- .01	0	- .01	- .01	- .02
16+80	0		0	0	- .02	- .01	- .03	- .01	- .01	- .02	- .03
17+40	+ .01				+ .01	- .02	- .01		+ .01	- .01	0
18+00	0		0	0	- .01	- .01	- .02	0	0	- .02	- .02
18+60	0	0	- .01	- .01	0	- .01	- .01	- .02	+ .01	- .02	- .01
19+20	0	- .02	+ .02	0	+ .01			- .02		0	
19+80		0	- .03	- .03	0	- .02	- .02	- .01	+ .01	- .03	- .02
20+40	- .01	0	- .01	- .01	+ .01	- .02	- .01	- .01	+ .01	- .02	- .01
21+00	- .02	0	- .02	- .02	0	- .02	- .02	- .01	0	- .02	- .02
21+60	- .01	+ .01	- .03	- .02	- .01	- .02	- .03	- .01	+ .01	- .01	0
22+20	- .03	- .01	- .03	- .04	0	- .02	- .02	0	+ .01	- .01	0
22+80	- .02	+ .01	- .01	0	0	- .02	- .02	- .01	0	- .01	- .01
23+40	- .01	- .01	- .02	- .03	+ .01	- .02	- .01	- .01	0	- .01	- .01
24+00	- .02	+ .01	- .03	- .02	0	- .01	- .01	- .02	+ .01	- .01	0
24+60	- .01	0	- .02	- .02	- .01	- .01	- .02	- .02	0	- .01	- .01
25+20	0	+ .01	- .02	- .01	0	- .02	- .02	- .02	0	- .02	- .02
25+80	0	+ .01	- .02	- .01	0	- .01	- .01	- .01	+ .01	- .01	0
26+40	0	0	- .02	- .02	- .01	- .01	- .02	- .02	0	- .03	- .03
27+00	0	+ .01	- .02	- .01	0	- .01	- .01	- .01	0	- .02	- .02
27+60	- .01	+ .02	- .02	0	- .01	- .01	- .02				
28+20	0	0	- .01	- .01	0	- .01	- .01	- .01			
28+80	0	0	- .01	- .01		- .01		- .02			
29+40	+ .02	+ .01	- .02	- .01				- .02	- .01	+ .01	0
30+00	+ .06	0	0	0							
30+60	+ .01	+ .01	0	0							
31+20	+ .01										
31+80	0										
32+40	+ .01										
33+00	+ .01										
33+40	+ .01										

ft with two 0.03 ft closures on the C-line. These closures are 1/6 to 1/2 of the 0.06 ft thick redwood board. The corner measurements on Fig. 25 show that the upper slab moves down toward the lower slab.

Leakage from the left abutment to Station 12+00 of the wing section is isolated from that from the rest of the dam. It reached a peak of 3.2 cfs for full reservoir and has steadily decreased. It was 0.9 cfs for reservoir level of 6520 ft elevation, the maximum level reached in 1959. In October, 1959, when the face of the wing section was exposed, cracks of about 1/32 in., paralleling portions of the abutment, were observed. They were chipped 1 in. deep and primed and filled with Igas mastic to prevent deterioration due to frost action more than to reduce seepage.

Upon initial filling, leakage from the main section was excessive. The source of leakage was located and underwater repairs were made to prevent loss of water. Some of the sloping joints had failed, possible due to a construction cause. The details of what actually occurred will be known when the Wishon Dam is unwatered through normal operation in a future year.

COURTRIGHT DAM

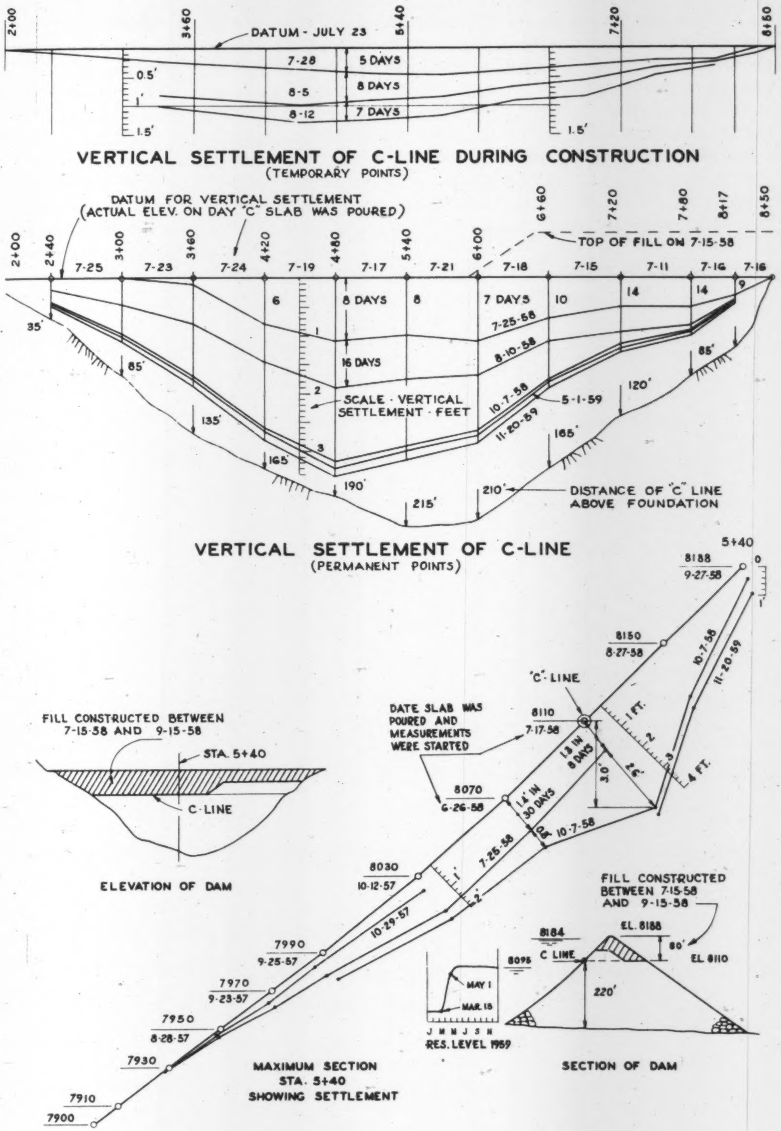
Courtright Dam was completed in October, 1958, several months after the paper was written. In 1959, the runoff was subnormal and the reservoir filled to only two-thirds the height of the dam. The available settlement data at this time is therefore limited. Fig. 26 presents a study of construction settlement and the movement of the dam due to the partial filling in 1959.

As shown on the elevation and section of dam on Fig. 26, the top 80 ft of rockfill was dumped between 7-15-58 (July 15) and 9-15-58 (Sept. 15). The two profiles of C-line vertical settlement and the maximum section showing settlement show the movements that took place during that period of construction, and also some movement before and after that period.

Vertical settlement of C-line during construction (temporary points) of Fig. 26 gives the movement of temporary points on the placed rock behind the slabs at C-line elevation. The readings are taken on different days than the readings on permanent points. The temporary point profile is shown only to confirm the permanent point readings that show such rapid movements during construction. The movements will be discussed with reference to the permanent points.

Vertical settlement of C-line (permanent points) of Fig. 26 gives the movement of each C-line point from the day the slab below C-line was poured to 11-20-59 (Nov. 20), about 1 yr and 4 months later. 3 ft of vertical settlement took place where the C-line was 220 ft above foundation during dumping of 80 ft of rockfill above the C-line as shown on Fig. 26. This is about 1.4%. The reason the movement, as measured, is greater at Station 4+80, than at the higher sections of 5+40 or 6+00, is that a small fill as shown above C-line was dumped before the measurements that are shown were started. In the year and one month between the 10-7-58 (Oct. 7) reading and the 11-20-59 (Nov. 20) reading, the maximum C-line movement was only 0.2 ft due to aging of the fill and the water rising 200 ft up on the 300-ft high dam and remaining there. This is significantly low compared to the 3-ft movement during construction. Of course, a substantial future movement will take place when the reservoir is first completely filled.

Maximum section, station 5+40,—showing settlement of Fig. 26 gives the settlement from the day each slab was poured until the various dates shown.


2+00
3+60
5+40
7+20
8+50
DATUM - JULY 23
7-28
5 DAYS
8-5
8 DAYS
8-12
7 DAYS
0.5'
1'
1.5'
1.5'
2+40
3+00
4+20
4+80
6+00
6+60
7+80
8+17
DATUM FOR VERTICAL SETTLEMENT (ACTUAL ELEV. ON DAY "C" SLAB WAS POURED)
TOP OF FILL ON 7-15-58
7-25
7-23
7-24
7-19
7-17
7-21
7-18
7-15
7-11
7-16
7-16
6
8 DAYS
8
7 DAYS
10
14
14
9
16 DAYS
7-25-58
8-10-58
10-7-58
5-1-59
11-20-59
SCALE - VERTICAL SETTLEMENT - FEET
1
2
3
35'
85'
135'
165'
190'
215'
210'
165'
120'
85'
DISTANCE OF "C" LINE ABOVE FOUNDATION
FILL CONSTRUCTED BETWEEN 7-15-58 AND 9-15-58
STA. 5+40
C-LINE
5+40
0
1
8188
9-27-58
8150
8-27-58
"C"-LINE
DATE SLAB WAS POURED AND MEASUREMENTS WERE STARTED
8110
7-17-58
1.3 IN 8 DAYS
1 FT.
2
3
4 FT.
3.0'
2.6'
8070
6-26-58
1.4' IN 30 DAYS
7-25-58
10-7-58
10-7-58
11-20-59
8030
10-12-57
1'
2'
10-29-57
7990
9-25-57
7970
9-23-57
7950
8-28-57
7930
7910
7900
FILL CONSTRUCTED BETWEEN 7-15-58 AND 9-15-58
EL. 8188
8184
C LINE
80'
EL 8110
8095
220'
MAY 1
MAR. 15
J M M J S N
RES. LEVEL 1959

VERTICAL SETTLEMENT OF C-LINE DURING CONSTRUCTION
(TEMPORARY POINTS)

VERTICAL SETTLEMENT OF C-LINE
(PERMANENT POINTS)

ELEVATION OF DAM

MAXIMUM SECTION
STA. 5+40
SHOWING SETTLEMENT

SECTION OF DAM

FIG. 26.—SETTLEMENT DATA—COURTRIGHT DAM

The reference line is the design location of the face and each row of slabs was poured to the design position. As the dam settled, the placed rock was steepened so as to return to the design lines and the top of the next row of slabs was poured to design grade. The bottom of the new slab was poured to the location of the lower slab. The settlement points are in the top of each slab and record the complete movement of the slab. While the C-point moved 2.6 ft during construction, the point 40 ft lower on the face moved only 0.8 ft and 40 ft still lower the movement was only 0.4 ft. The distance between the reference line and the 10-7-58 line gives the total construction settlement from the time the slab was poured. The distance between the 10-7-58 (Oct. 7) line and the 11-20-59 (Nov. 20) line indicates the small settlements caused by the first years aging and by filling to two-thirds the height of the dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3084

ROCKFILL DAMS: DERBENDI KHAN DAM

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With Discussion by Messrs. S. Sakurai; and Calvin V. Davis

SYNOPSIS

Described in this paper are the problems pertaining to the design of a 450-foot high central core rockfill dam. Foundation excavation and preparations for diversion are now under way. The Derbendi Khan project consists of the rockfill storage dam, chute spillway, outlet works, and provisions for a future powerhouse. Diversion of the river was provided for by two tunnels that will serve the permanent outlet works and powerhouse after construction.

A thorough field exploration and materials testing program provided a sound basis for the selection and design of the dam. Major problems were anticiapted and are being provided for as a result.

GENERAL DESCRIPTION

The Derbendi Khan rockfill dam is located on the Diyala-Sirwan River approximately 230 kilometers northeast of Baghdad, Iraq, and ten kilometers downstream from the confluence of the Tanjero River. The project is owned and will be operated by the Development Board of the Government of Iraq. The location of the project and its drainage are shown in Figure 1. At the Derbendi Khan site the river flows through a three-kilometer long gorge in the Baranand Dagh mountain range. This range is part of the northwest-southeast chain of mountains forming the Persian arc. The site is located about one kilometer above the downstream end of the gorge.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1741. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The primary function of the Derbendi Khan dam will be the storage and regulation of the Diyala-Sirwan River for irrigation use. The greatest value and need for water in Iraq is for agricultural irrigation in the fertile but dry plains of the Tigris and Euphrates valley. Also the regulation of the river and the head created by the dam will make feasible the generation of hydroelectric power at Derbendi Khan as a secondary benefit of the project. Some measure of flood control will be available at Derbendi Khan, although not without slight reduction in irrigation storage benefits. As presently planned, the flood control is considered of secondary importance.

Above the dam site the Diyala-Sirwan River has a drainage area of about 17,850 square kilometers (6900 square miles). The 30-year average annual flow passing the dam site is 3.8 milliards of cubic meters (3,100,000 acre-feet). The gross storage of the Derbendi Khan dam is 3.0 milliards (2,430,000 acre-feet). A typical hydrograph is shown on Figure 2. The maximum flood of record is very close to 4000 cubic meters per second (141,500 cfs). During the months of January through May the river flows are generally in excess of 200 cubic meters per second (7100 cfs). The flows from June through December are usually less than 100 cubic meters per second dropping to less than 50 cubic meters per second in November and December.

Figure 3 is a plan of the major structures of the project. The rockfill dam extends from the left abutment across the river to the right abutment, where the intake tower, guide wall, and the chute spillway are located.

Two circular concrete-lined tunnels provide for diversion around the right end of the dam. The tunnels are six meters (19.7 feet) and nine meters (29.5 feet) in diameter and after they have served for diversion they will be converted so they may be used for irrigation and power releases. This will be accomplished by extending shafts from the tunnels to the intake tower at upstream end of the tunnel. At the downstream end a transition will be placed and smaller diameter penstocks will branch off and connect with the outlet works and future powerhouse. Steel liner will be used in portions of the six-meter and nine-meter tunnel and the penstocks.

The hydraulic configuration of intake tower, guide wall, and chute spillway was established as the result of model tests carried out at the St. Anthony Falls Hydraulic Laboratory under the direction of Dr. Lorenz G. Straub. Model studies of the initial designs of the intake tower and guide wall indicated minor disturbances in the flow approaching the spillway. However, this was enough to generate a series of wave patterns and led to unstable flow conditions in the chute. Alterations of the guide wall, streamlining of intake tower, and leveling the approach channel floor solved this problem. The bucket at the downstream end of the chute is designed to turn the water away from the slope and into the stream bed downstream from the concrete structures. At

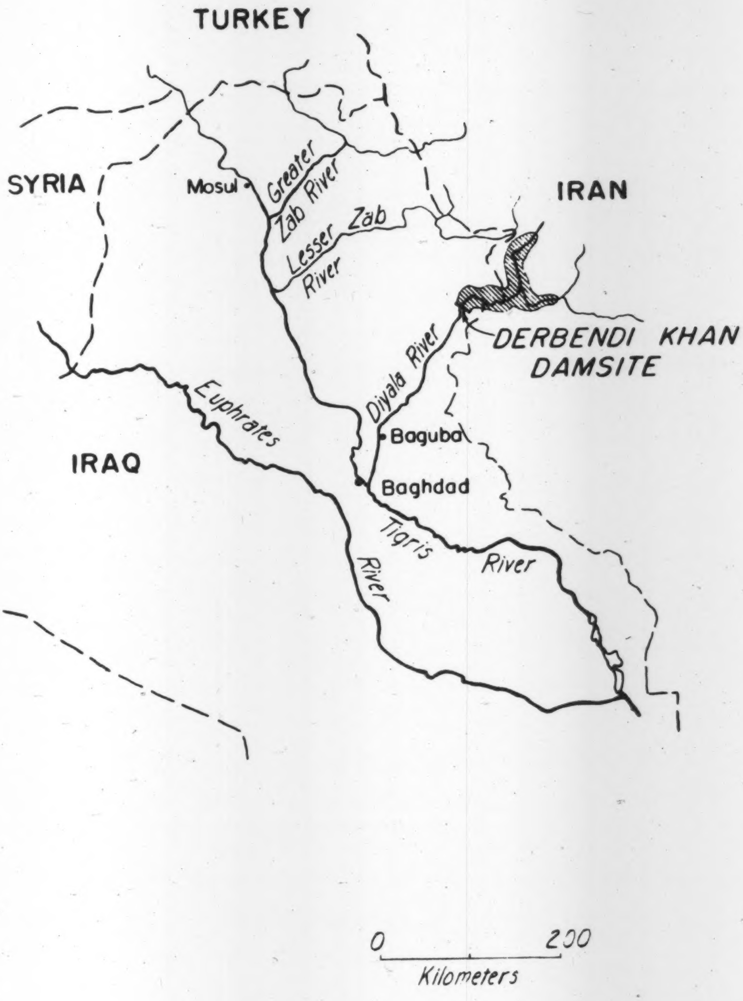

TURKEY
SYRIA
IRAN
IRAQ
Mosul
Greater Zab River
Lesser Zab River
DERBENDI KHAN DAMSITE
Diyala River
Baquba
Baghdad
Euphrates River
Tigris River
0
200
Kilometers

FIG. 1.—LOCATION MAP

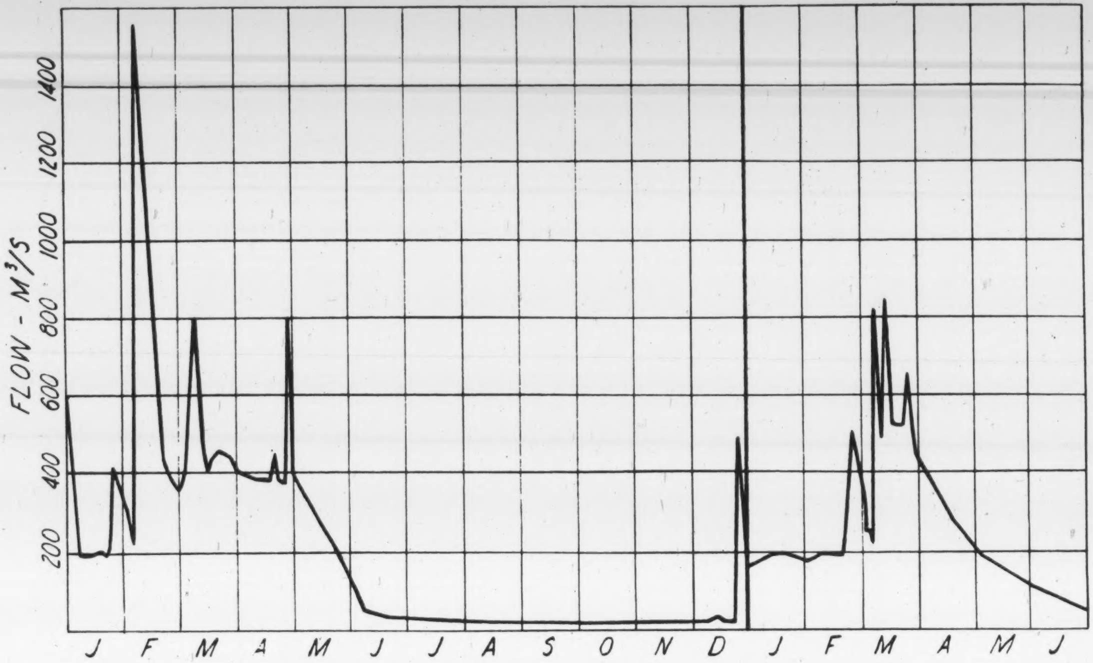


FIG. 2.—TYPICAL HYDROGRAPH, 1941 AND 1942, DIYALA RIVER, DISCHARGE SITE

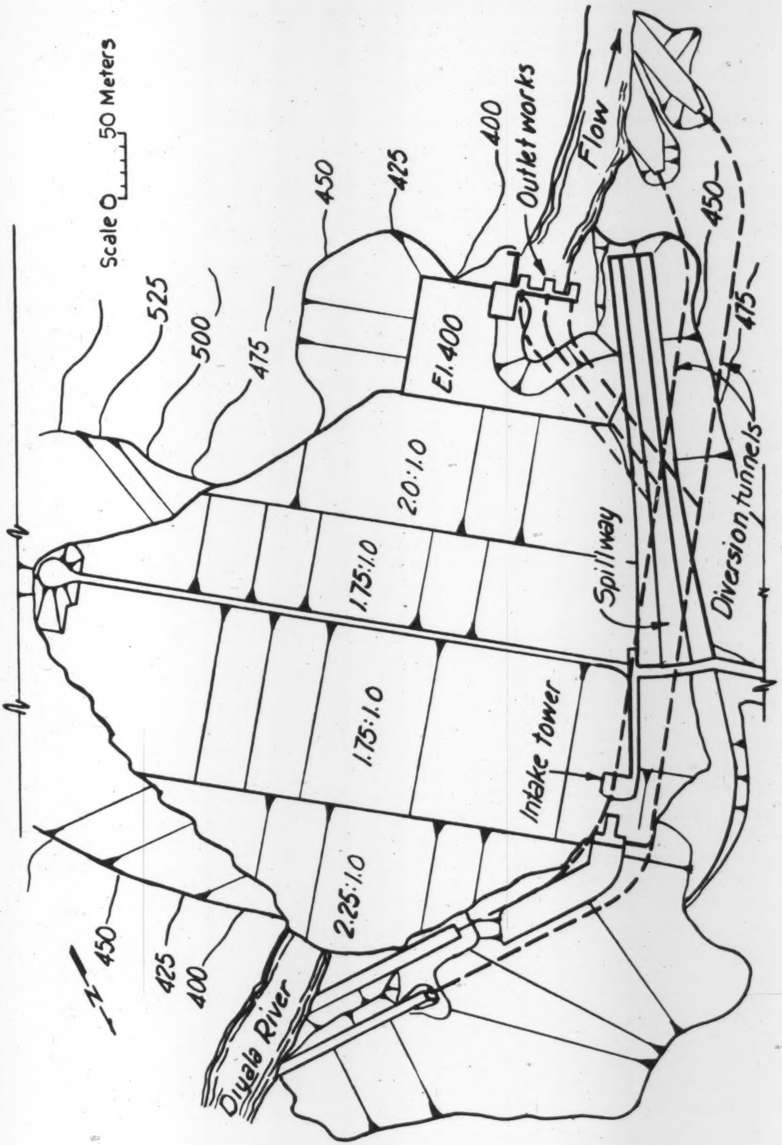

Scale 0 50 Meters
N
525
500
475
450
425
400
Outlet works
Flow
El. 400
2.0:1.0
1.75:1.0
1.75:1.0
2.25:1.0
Spillway
Diversion tunnels
Intake tower
450
475
450
425
400
Diyala River

FIG. 3.—DERBENDI KHAN PROJECT PLAN

maximum discharges the jet will land in the stream bed about 150 meters downstream from the bucket.

Three outlet valves of the fixed-cone dispersion type, 2.438 meters (96 inches) in diameter, will be used to regulate the irrigation releases. The valves are hooded to control the degree of dispersion.

Initial construction of the power facilities is limited to a pair of 1000 kva generating units for operating the gate hoists, outlet valves, and other electrical equipment on the dam and also supply the housing facilities for the operational staff. The arrangement of the outlet valves and the service generating unit is such that two larger units with a total capacity of 37,000 kw can be installed in the future.

Geology of the Site

The Derbendi Khan dam site is situated where the Diyala-Sirwan River has cut a deep gorge through a northwest-southeast trending ridge known as the Baranand Dagh. This ridge is composed of a series of thick beds of sedimentary rocks including marls, sandstones, limestones, and conglomerate. Deposited over 100 million years ago, these materials have had a varied geologic history. About ten million years ago the beds were compressed by horizontal forces which folded them into a series of anticlinal ridges.

The anticlinal ridges have great economic significance in the Middle East, because their dome-like shape has served to trap the tremendous oil resources of that area. However, the Baranand Dagh anticline was cut by the river and eroded, allowing the oil to escape. In the immediate vicinity of the dam site, this history is evidenced by the presence of bitumen, which impregnates some of the formations.

The geologic formations which occur in the foundation area are a 335-meter thickness of bituminous marl, a 200- to 300-meter thickness of a sandstone-marl-limestone complex, identified as the Buff formation, and a 70- to 150-meter thickness of green marl of varying soundness. Overlying the green marl is a fine grained, dense limestone, usually massively bedded. The dip of the strata at the dam site ranges from 35 degrees to over 80 degrees in the downstream direction.

The bituminous marl formation consists mainly of calcareous shales with lenses of calcareous sandstone and argillaceous limestone. The Buff formation consists of interbedded lenses of limestone, shale, sandstone, and conglomerate. The green marl formation consists of calcareous shale, conglomerate, and interbedded lenses of sandstone, shale, and limestone. All three formations can be identified as calcareous and of the compaction type. Thus it is not uncommon to encounter substantial areas and thicknesses which may disintegrate gradually when submerged in water and then exposed to the atmosphere. The compressive strengths of core samples indicated a wide variation of strength in each formation. Table 1 shows the test results and allowable bearing pressures.

The dam site is on the downstream limb of the anticline, where a local cross-flexure has ruptured the strata. Approximately 200 meters downstream from the toe of the dam this cross-flexure passes into a small fault which follows the river bed. This fault does not extend into the area of the dam site and there are no reasons to believe it is active.

TABLE 1.—ROCK FORMATIONS, TEST RESULTS AND ALLOWABLE BEARING PRESSURES

Formation	Description	Unconfined Compression Tests, in Tons per Square Foot		Allowable Bearing Pressure, in Tons per Square Foot
		Minimum	Maximum	
Upper Green Marls	Compacted, calcareous, clayey shale.	140	500	Not used as a foundation rock
Lower Green Marls	Shale, sandstone, and sandy limestone.	445	1030	25
Buff "D"	Limestone, sandy limestone, sand stone, and shale.	270	1070	25
Buff "C"	Nodular limestone with lenses of shale and sandstone.	190	1000	25
Buff "B"	Clayey and sandy shale, with beds of nodular limestone.	85	515	10
Buff "A"	Calcareous sandstone with conglomerate lenses.	660	1025	25
Bituminous Marls	Compacted calcareous and argillaceous shale, with beds of nodular limestone.	9	265	7.5

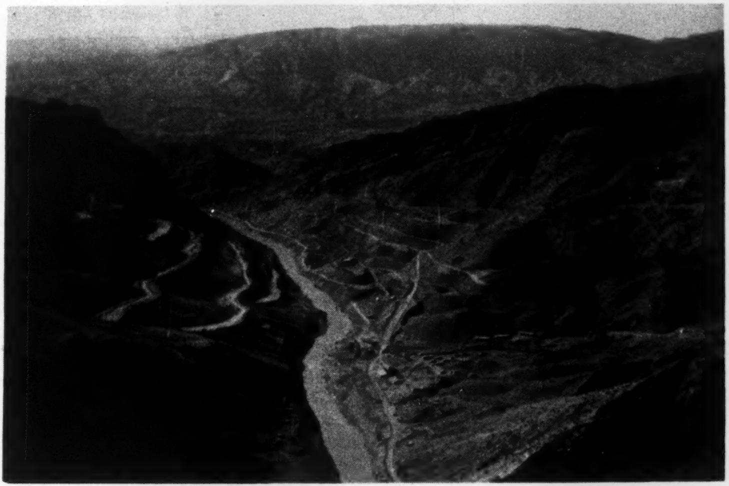


FIG. 4.—VIEW LOOKING DOWNSTREAM DERBENDI KHAN PROJECT

The gorge topography of the Derbendi Khan site is emphasized by the nearly vertical cliffs of Qarah Chauq limestone, which form the top of the gorge walls on both abutments above the dam and spillway and can be noted in Figure 4 From the foot of the limestone cliffs talus slopes extend down to the river banks. The talus has in almost all areas weathered to a soil which supports sparse vegetation to the extent permitted by the semiarid climate.

An analysis of the geologic and topographic conditions was the basis of the final selection of the axis of the rockfill dam and spillway. Of the aforementioned succession of formations, the bituminous marl is under the upstream half of the rockfill dam and for a slight distance under the core in the river bed. The Buff formation in general underlies the core, the upstream shell at higher levels of the abutments, and the entire downstream shell of the dam, as indicated in Figure 5. The intake tower is also located within the Buff formation on a 20-meter thickness of sandstone. The guide wall, spillway ogee, and the upstream portion of the spillway chute are located on the marls and limestones of the Buff formation. The downstream portion of the spillway chute and the terminal bucket are located on the more competent portions of the green marl.

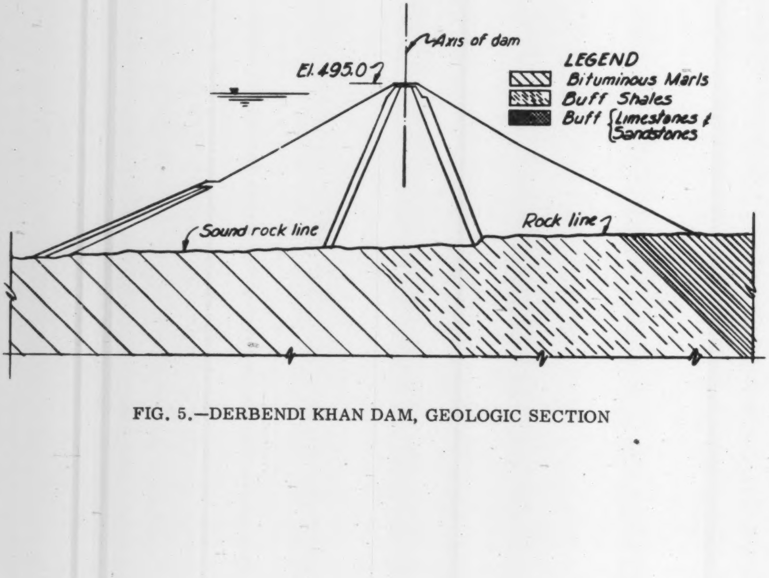


FIG. 5.—DERBENDI KHAN DAM, GEOLOGIC SECTION

On the left abutment the overburden, formed by the weathered talus, ranges in depth from about two to 55 meters, the average depth being in the order of ten to fifteen meters. The overburden consists usually of debris from the Qarah Chauq limestone in a matrix of clay. The limestone debris ranges from fragments a few centimeters in diameter to blocks over five meters in diameter, generally separated by the clay matrix. In its natural state the matrix is dry and hard and the overburden, as a mass, is relatively impermeable.

On the right abutment the overburden varies considerably in character and thickness depending on the underlying rock formation. In the areas underlain by the bituminous marl the overburden is generally a clay soil derived from the marl with isolated blocks of limestone, usually less than one meter in diameter. The overburden varies in thickness from eight meters to 38 meters.

In the areas underlain by the Buff formation the limestone debris from the overshadowing cliffs is predominant, forming a limestone talus in which the interstices between the limestone particles are filled with clay and silt making the deposit relatively impermeable. The talus has a maximum thickness of approximately eight meters at the base of the slope.

In the area underlain by the green marl formation, the overburden is a green residual clayey soil with isolated blocks of limestone usually less than one meter in diameter. This soil is very plastic when wet and there is evidence of creep during the rainy season. Thickness is estimated to average eleven meters.

In the river channel the deposits consist primarily of blocks of limestone and other resistant rocks ranging up to seven meters in diameter. Explorations indicate that the river deposits will not be greater than few meters thick on the average.

Design of Dam

In the preliminary studies of the Derbendi Khan site considerations of the various alternative types of dam, which might be suitable for the foundation conditions, were narrowed to the studies of a rockfill and concrete gravity type dam. Comparative cost estimates tended to favor the rockfill type, but in an area where price precedence had not been established by previous construction, the magnitude of the estimated cost difference was not sufficient to warrant elimination of a concrete dam study. Accordingly the designs for the rockfill dam and the concrete gravity dam were carried along through the various preliminary stages including the preparation of contract drawings and specifications for both.

Tenders were published in April 1955 for both a rockfill dam and concrete gravity dam. The bids were received and opened on August 31, 1955. After thoroughly analyzing, interpreting, comparing, and evaluating the complex pattern of bids from six contracting combines, it was found that the rockfill structure was nearly fifteen percent cheaper than the lowest bid for the concrete gravity dam, representing a savings of approximately eight million dollars. This substantial differential in favor of the rockfill dam eliminated any further consideration of the concrete gravity dam.

During the preliminary studies of the rockfill shell with impervious core type of structure, conferences and consultations led to the following conclusions:

1. The unquestionable safety of the dam is of utmost importance, hence the design should reflect a reasonable degree of conservatism. The principal factors dictating such caution were the height of the dam, the heterogeneous nature of the foundation materials, and the somewhat limited area of strong rock, upon which to construct the spillway.

2. After a consideration of sloping core and central impervious core types of rockfill dams, the central core type was selected. The reasons for this selection were based on:

a. The low friction strength of the available core material would necessitate flatter slopes for the upstream shell of a sloping core type dam than for a central core type dam. Thus for a given upstream slope the central core type had a greater upstream stability. Flattening the upstream slope would increase the cost of the intake structure and guide wall.

b. In order that the major portion of the core be in contact with the Buff formation, only a central core could be adopted.

c. The central core contact area with abutment is greater than a sloping core. Also the full weight of the dam is applied to the contact area providing maximum safety against seepage. The pressure exerted along the foundation line of a sloping core is considerably less.

The above factors were very desirable in a more than 400-foot high rockfill dam.

3. Core side slopes of about 0.3 horizontal to 1.0 vertical were selected in preference to thicker sections. A thicker core would require additional shell material to provide an equivalent factor of safety against sliding under the condition of sudden drawdown.

4. The base of the impervious core section and at least the first or finest of the filter zones would be carried to sound and impervious rock throughout the entire foundation.

5. Pending exposure of the excavated foundation, the use of a small narrow cutoff trench excavated along the line of the proposed grout curtain was considered unnecessary; likewise, the pouring of a concrete cap to provide some support against uplift grouting pressures during first stage grouting was deemed unnecessary.

6. It was agreed that the downstream stability section of the dam would consist essentially of dumped quarry-run rock with minimum average downstream slopes of 1.75 to 1 for the upper two thirds of the slope and 2.0 to 1.0 for the lower one third of the downstream slope. It was understood that the slopes would be reviewed after stability computations of the relative safety against sliding along the foundation were completed.

7. The slope or slopes of the upstream section would be determined by stability calculations, such calculations to take into account resistance to sliding along the contact of the rockfill shell and the shale foundation as well as sliding within the embankment shell. These computations were based on use of quarried rock or stream bed snads and gravels for the shell material.

8. The crest section as wide as dictated by construction requirements.

Starting with the previously described criteria the resulting rockfill section was analyzed for the following conditions:

Case I. Construction condition: embankment completed to maximum elevation 495 meters and no water in the reservoir.

Case II. Construction condition: top of embankment at elevation 423 meters as required for diversion during the wet season, and with no water in reservoir.

Case III. Normal operating condition: reservoir water level at top of gates, elevation 485 meters.

Case IV. Rapid drawdown: normal operating level elevation 485 meters drawdown to maximum drawdown level elevation 434 meters.

Case V. Normal operating condition: reservoir at elevation 485 meters with the added forces due to a horizontal earthquake acceleration of 0.10 gravity.

Case VI. Catastrophic flood condition: all gates raised and the reservoir level within 1.5 meters of the top of the embankment, pool at elevation 493.5 meters.

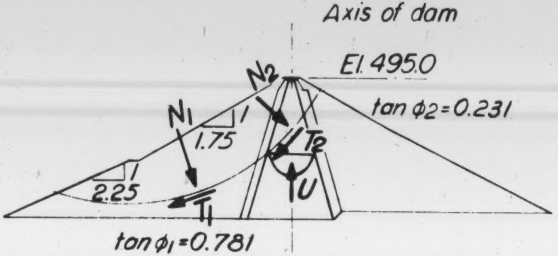
The maximum section at the river and other critical sections at the abutments were analyzed for stability. The analyses were made by the "sliding wedge" and "slip-circle" methods. Of these two methods the "sliding wedge" analysis is considered most applicable and will present more accurately the stability of a central core type rockfill dam. A "slip-circle" analysis will give a higher factor of safety under similar loading conditions. This can be explained by noting that the slip-circle only passes through a small portion of the core material and thus does not reflect the effect of the high pore pressures in the core on the stability of the shell material. A comparison of Figures 6 and 7 will show that the "sliding-wedge" analysis takes into account the full effect of the driving force created by the pore pressures in the core.

The most critical situation was with loading Case V with reservoir at elevation 485 and earthquake acceleration equal to 0.10 gravity, in which instance the downstream slope when analyzed by the sliding-wedge method showed a minimum factor of safety of 1.22. Factors of safety for the upstream and downstream slope analyzed for various loading cases are tabulated in Table 2. Considering the extreme infrequency of the loading defined in Cases V and VI, the safety factor values of 1.22 and 1.30, respectively, were considered acceptable. A factor of safety of 1.47 for the downstream slope under normal operating conditions (loading Case III), may be considered as an index of the reserve stability for normal operating conditions.

After minor modifications to foundation conditions, diversion requirements, and the limitation set by the appurtenant structures, the maximum section at the river bed was established as shown in Figure 8.

The upstream shell and the impervious core excavation will be carried down to sound rock while the downstream shell will be permitted to rest on slightly weathered rock. The exposed slopes of overburden excavation will be 1.35 horizontal to 1.0 vertical. To minimize the excavation volume a five-meter thickness of overburden was permitted to remain on the left abutment near the top of the dam. As the dam is built in this area the overburden will be removed permitting placement of the fill on sound rock. The excavation will be carried out in benches as shown on Figure 9.

Because some of the marl areas were susceptible to drying and checking when exposed to the atmosphere for long periods of time it was specified that all marl foundations be coated with a bituminous sealer immediately after excavated to grade. Area under the impervious rolled earth core is to be treated with a slush grout in lieu of the bituminous sealer. In cases where the slope of the foundation is too steep for application of slush grout the exposed rock will be gunited to a minimum thickness of fifteen millimeters.


Axis of dam
El. 495.0
N2
N1
1
1.75
tan ϕ_2 = 0.231
T2
U
1
2.25
T1
tan ϕ_1 = 0.781

Driving forces:

$$\Sigma T = 11610^K$$

Resisting forces:

$R_1 = \Sigma N_1 \times \tan\phi_1$ $R_1 = 14770^K$

$R_2 = \Sigma(N_2 - U)\tan\phi_2$ $R_2 = 678^K$

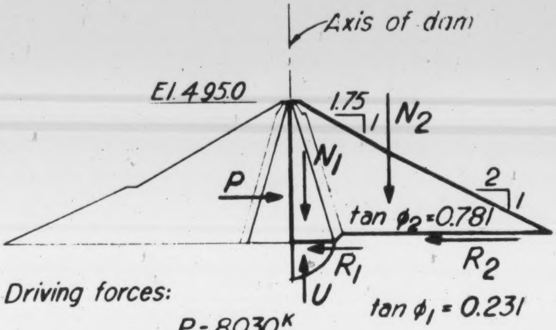
$cL = 972^K$

16420^K

Factor of safety:

$$F.S. = \frac{16420}{11610} = 1.41$$

FIG. 6.—CASE IV, DRAWDOWN FROM EL. 485 TO EL. 434


Axis of dam
El. 495.0
1.75
1
N2
N1
P
2
1
tan ϕ_2 = 0.781
R1
R2
U
tan ϕ_1 = 0.231

Driving forces:

$P = 8030^K$

$U = 2215^K$

Resisting forces:

$R_1 = (N_1 - U)\tan\phi_1$ $R_1 = 1345^K$

$R_2 = N_2 \times \tan\phi_2$ $R_2 = 10060^K$

11405^K

Factor of safety:

$$F.S. = \frac{11405}{8030} = 1.42$$

Min. F.S. = 1.22 (Earth quake)

FIG. 7.—CASE III, WATER AT EL. 485

TABLE 2.—DERBENDI KHAN PROJECT, STABILITY ANALYSIS, MAXIMUM SECTION AT RIVER

SLOPE	LOADING CASE	METHOD ANALYSIS	SAFETY FACTOR
UPSTREAM	I	SLIP CIRCLE	2.21
		SLIDING WEDGE	1.41
	II	SLIP CIRCLE	1.82
		SLIDING WEDGE	1.47
	IV	SLIP CIRCLE	1.41
DOWNSTREAM	I	SLIDING WEDGE	1.83
	III	SLIDING WEDGE	1.47
	V	SLIDING WEDGE	1.22
	VI	SLIDING WEDGE	1.30

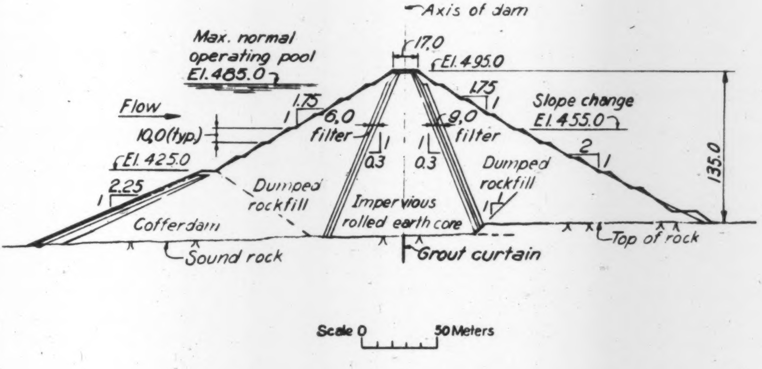

Axis of dam
Max. normal operating pool El. 485.0
17.0
El. 495.0
1.75
1
Slope change El. 455.0
Flow
1.75
1
6.0 filter
9.0 filter
10.0 (typ.)
1
0.3
1
0.3
El. 425.0
2
1
135.0
2.25
1
Dumped rockfill
Dumped rockfill
Impervious rolled earth core
1
1
Cofferdam
Sound rock
Grout curtain
Top of rock
Scale 0
50 Meters

FIG. 8.—SECTION THROUGH DERBENDI KHAN DAM

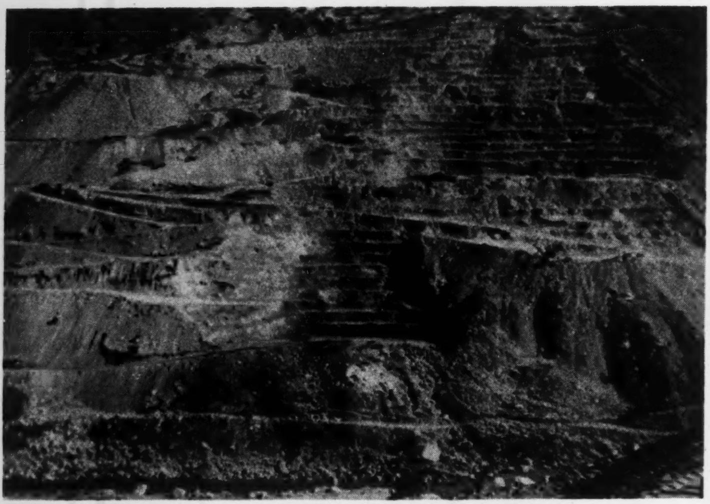


FIG. 9.—LEFT ABUTMENT EXCAVATION, DERBENDI KHAN PROJECT

To facilitate construction of the dumped rockfill shells, the exterior slopes may be formed in ten-meter benches as shown in Figure 10. The width of these benches is designed so that the average slope, i.e. from the center to center of each bench, will be equal to the design slope. The slope between each bench will be the natural dump slope of 1.35 horizontal to 1.0 vertical.

To provide maximum compaction of the dumped rockfill shell the rock will be end-dumped and sluiced with hydraulic monitors. The monitors will be directed so that fines are forced down into the voids between the larger rocks. This will reduce the voids and keep future settlements to a minimum. The ratio of sluicing water volume to rockfill is specified as three to one. The lifts for end dumping are specified as 30 meters (100 feet) maximum and ten meters minimum.

The transition from the fine core material to the coarse rockfill is specified in two stages on the upstream side and three stages on the downstream side. Each filter stage is three meters wide to permit placement by end dumping from trucks and spreading in layers by mechanical methods. The filters are to be compacted along with core material and in the same manner.

The core material is to be placed in 30-centimeter or 15-centimeter layers and compacted with either rubber tired or sheeps foot rollers, respectively. The moisture content is to be controlled to develop the maximum practicable density with four passes of the compaction equipment.

The grout curtain is shown on Figure 10. The deep cutoff grout curtain extends from the left abutment of the dam to the spillway structure on the right bank where it connects with the grout curtain under the gallery in the crest structure of the spillway. The grout curtain terminates beyond the right end of the spillway at elevation 495.0 meters (the crest of the dam). The stage grouting method by zones will be used to develop the grout curtain. Each hole is to be washed and pressure tested prior to the introduction of grout. Table 3 shows the specified grouting criteria for each stage. As each stage of grouting is completed the pressures for the succeeding stage may be increased. Stage I serves primarily as a low pressure, foundation consolidation operation.

TABLE 3.—GROUTING CRITERIA

Stage	Depth in Meters	Maximum Pressure	Design Spacing Meters
I	6	1.0 kg/cm^2	1.60±
II	15	3.5 kg/cm^2	1.60±
III	30	7.0 kg/cm^2	1.60±
IV	60	7-14 kg/cm^2	6.50±
V	90	14-21 kg/cm^2	—

In the foregoing tabulation Stages II and III are further extensions for the grout holes started as Stage I. Stage IV constitutes an exploration and a check by extending approximately every fourth hole of the Stage III pattern. Stage V comprises random spaced check holes, primarily for further exploring leaky areas detected by stage operations.

Construction Materials

Exploration for suitable construction materials revealed that soil suitable for use as an impervious core, quarriable rock for constructing the shells of the dam and a supply of sand and gravel which could be processed for use as filter material were available within economic haul distances. The soil borrow area is located on a plateau approximately 1-1/4 kilometers from the dam site at an elevation of about 400 meters above sea level. A series of test pits defined the soil profile as having an average of ten centimeters of topsoil below which there is a zone of loamy sand averaging 0.6 to 0.8 meters in thickness, which in turn is underlain by a third zone similar to the loamy sand, except for the presence of calcium carbonate nodules formed by reprecipitation of the calcareous materials leeched from the overlying zones. Our present plans are to use the second zone with an average thickness of 0.7 meters. It may well be that after the borrow area is opened some area of the third zone may not have an excessive percentage of calcareous nodules. If such is the case, these materials may be used for the impervious core.

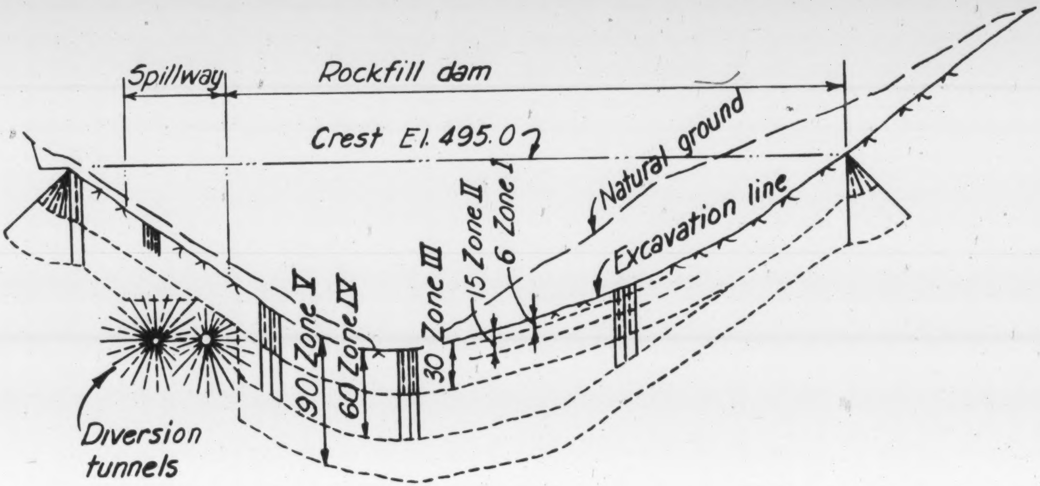

Spillway
Rockfill dam
Crest El. 495.0
Natural ground
Excavation line
6 Zone I
15 Zone II
30 Zone III
60 Zone IV
90 Zone V
Diversion tunnels

FIG. 10.—SECTION ALONG GROUT CURTAIN, DERBENDI KHAN DAM

Laboratory tests of the impervious borrow material indicate that it has a natural moisture content of 22 percent. The moisture content does not appear to decrease during the dry season. Laboratory compaction tests of the borrow material disclosed an average optimum moisture content of 14.5 percent and a corresponding average dry density of 116.5 pounds per cubic foot. Triaxial tests of the compacted borrow material were made and the results incorporated in the final design of the dam. The results indicated that an average friction value of thirteen degrees and cohesion of 1.1 kg/square centimeter could be used. Approximately 80 percent of the borrow material passed a 200 sieve and when compacted provided a low coefficient of impermeability. Further testing revealed that compaction of material with moisture contents above optimum would result in the development of high pore pressures. Therefore, field moisture control must carefully be followed and methods developed to reduce the natural moisture content prior to compaction in the impervious core.

The steep walls of the Derbendi Khan gorge are favorable to quarrying operations and quarries can easily be developed on either bank within a kilometer or so of the dam site.

The Qarah Chauq limestone which forms the steep walls of the gorge is a moderately hard, dense to porous rock suitable for the shell of a rockfill dam or, when crushed, for concrete aggregate. A test area on the right bank was blasted to explore the quarriability of the material and the manner in which it would break when blasted. The results of the test blasts were extremely satisfactory. Generally, the blast produced no boulders larger than 0.2 cubic meters and retrievable sizes were well assorted from four to 75 centimeters. There were some slabby and splintery fragments in the smaller sizes, but inasmuch as these fragments will settle into the voids between the larger blocks the percentage was not considered objectionable. Bonding surfaces were clean and rough and the amount of fines was not excessive. Compression tests on cores of the Qarah Chauq limestone gave values in the 800 to 900 kilogram per square centimeter (11,500 psi) range at failure.

Suitable sources of gravel and sand are located on the Diyala-Sirwan River with the major deposit approximately five kilometers upstream from the dam site. It is estimated that at least four million cubic meters are readily available in a deposit several hundred meters wide and over a kilometer long. This is more than ample for use as concrete aggregate and filter material. The deposits consist of almost unstratified and practically unconsolidated alluvial and talus materials derived mainly from limestone with some chert, basic igneous and metamorphic rocks. These materials contain some 40 percent sand with grading reasonably satisfactory for use in concrete,but will require washing since the proportion of clay and silt is fairly high.

Construction

Major construction started in February 1956. The initial operations were completion of the access roads, construction plant preparations, excavation and completion of the diversion facilities.

Some of the basic unit bid prices for construction at the time of award are listed below:

TABLE 3.—SIGNIFICANT DATA, DERBENDI KHAN ROCKFILL DAM

Drainage Area	17850 sq km	6883 sq miles
Length of Reservoir	30 km	18.6 miles
Maximum Height of Dam	135 m	443 ft
Width of Base	513 m	1684 ft
Crest Length	445 m	1460 ft
Total Volume of Fill	7,000,000 cu m	9,156,000 cu yd
Material Characteristics		
Impervious Core, θ = 13°, Cohesion = 1.1 kg/sq cm		
Dumped Rockfill, θ % 38°		

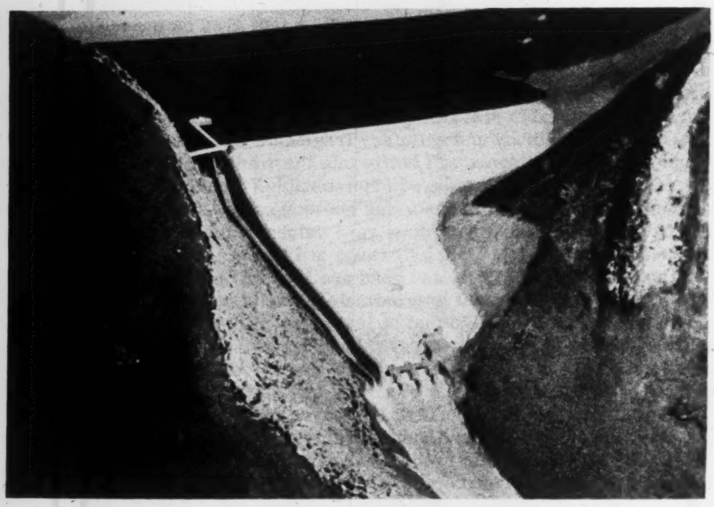


FIG. 11.—MODEL OF COMPLETED DERBENDI KHAN PROJECT

Item	Unit Price/cubic meter
Rockfill	$1.75
Filters	$4.80
Core Material	$1.75
Open Common	$0.65
Excavation	
Open Rock	$2.30
Concrete (Including Cement, Forms, and Reinforcement)	$30.00 (Average All Classes)

The initial diversion scheme planned on the use of a low cofferdam (incorporated in the upstream shell with an impervious blanket) for dry season closure. This plan provided sufficient time to construct the upstream shell and central core to elevation 425 meters with sufficient downstream shell for stability. Thus, in this scheme the main core would serve as the wet season cofferdam and enable flows of 1600 cubic meters per second to pass through the two diversion tunnels. Including the effect of storage upstream of the cofferdam, floods of 2600 cubic meters per second could be handled without overtopping.

However, construction difficulties delayed the start of diversion and the initial wet season cofferdam scheme was modified. The present cofferdam scheme is shown on Figure 10. The effect of the modification was to reduce the amount of material to be placed prior to wet season diversion and thus minimize the delays.

SUMMARY

A tabulation of the salient features of the Derbendi Khan project is given in Table 4. A photograph of a model showing the completed project is shown on Figure 11. The model indicates the anticipated extent of the excavation above the dam.

The Development Board of Iraq through their First Technical Section have administrated the development of the Derbendi Khan project.

The dam is under construction by the Derbendi Khan Contractors, a joint venture headed by the J. A. Jones Construction Company of Charlotte, North Carolina. The hydraulic model studies of the spillway and outlet works were performed by Dr. Lorenz G. Straub, M. ASCE, at the St. Anthony Falls Laboratory of the University of Minnesota. Mr. I. C. Steele served as Consultant on the foundation and rockfill dam design. Mr. D. W. Griffiths, Affiliate Member ASCE, is serving as Resident Engineer at the project. Mr. D. P. Roberts, M. ASCE, served as the Project Manager in early stages of design. Mr. E. J. Beck, A. M. ASCE, is presently serving as Project Manager.

DISCUSSION

S. SAKURAI.[2]—The writer has read the author's paper with particular interest since he is engaged on the Makio Dam, of similar type and magnitude to Derbendi Khan Dam. The writer will summarize the site conditions and the solutions at Makio.

Makio Dam is a multi-purpose project located on the Otaki River, near Nagoya, Japan. The dam is of thin central core and alluvial-rockfill shell type, of 350 foot height, as shown on Fig. 12. Construction began in late 1957, embankment construction will begin in July of 1959, and completion is scheduled in December of 1960.

Geology

The formation consists of strata of sandstone, slate and chert of Paleozaic, and diabase inserted in some slate. The strike is in general up-downstream direction and dip 30° - 70°. There is a fault at the center of the river and several smaller faults of crushed material and clay, A, B and C of Fig. 12(c). There are four steps, each several meters thick, of terrace materials of andesite and granite sands and gravels. The river bed deposit is 25 meters thick with extensive boulders near the ground surface and more sands than gravel at the bottom. Nearly 100% CO_2 gas and a little H_2S gas is emitted at the site and ground waters contain free CO_2 in nearly saturated condition, $Ca\,O$, $Mg\,O$, Fe_2O_3, $Si\,O_2$, and $A\ell_2\,O_3$ as dissolved from the rocks. Gas and low pressure springs are general in the area.

A pervious saddle near the damsite, consisting of an ancient channel of alluvium, volcanic ash and pumice, was thoroughly investigated and accepted.

Construction Materials

Materials conveniently available for embankment construction are: rock from spillway excavation (sandstone); transition material of well graded sand-gravel-cobble-boulder alluvium; core material from downstream river terraces (5 - 15 m. thick) of volcanic ash and breccia of andesite, chert, slate, sandstone, etc., and filter material from selective borrowing in the above alluvium deposits.

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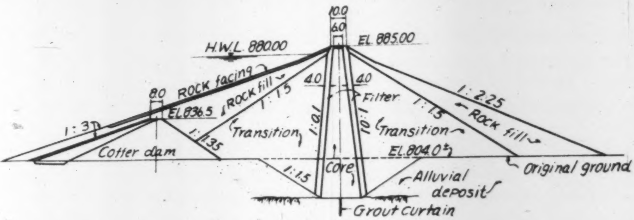
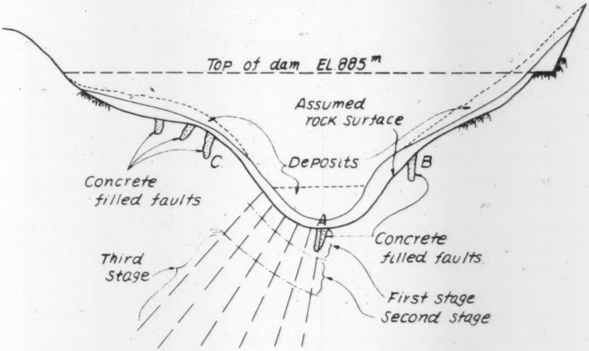
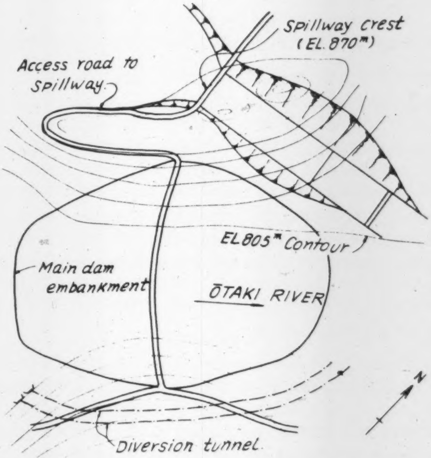
[2] Dir., Aichi Irrigation Public Corp., Nagoya, Japan.


10.0
6.0
H.W.L. 880.00
EL. 885.00
8.0
ROCK facing
ROCK fill
4.0
4.0
Filter
1:2.25
EL.836.5
1:1.5
1:0.1
0.1:1
1:1.5
Rock fill
1:3
Transition
Transition
Cotter dam
1:1.35
EL.804.0±
Original ground
1:1.5
Core
Alluvial deposit
Grout curtain

(*a*) EMBANKMENT SECTION


Top of dam EL.885 m
Assumed rock surface
Deposits
C
B
A
Concrete filled faults
Concrete filled faults
Third Stage
First stage
Second stage

(*b*) LONGITUDINAL SECTION


Spillway crest (EL.870 m)
Access road to Spillway
EL805 m Contour
Main dam embankment
ŌTAKI RIVER
N
Diversion tunnel

(*c*) PROJECT PLAN

FIG. 12.—MAKIO DAM

Core Material

The core material is unusual and of special interest. Grains of size less than 2 mm consist of volcanic ashes and a small amount of flakes of breccia and small pieces of pumice or talus. Classification is sandy loam and color yellowish brown. Specific gravity is 2.7 to 2.8 and grading is 5 - 15% Clay (0 - 0.005 mm), 10 - 34% silts (0.005 - 0.05 mm), 15 - 35% fine sands (0.05 - 0.25 mm), 10 - 25%, coarse sands, and 5— 10% gravels. Natural dry density is 1.0 to 1.1 ton/m^3 (63 to 69 lb/cu ft) for material under 4.8 mm grain size and natural moisture content ratio 60%. With samples including 150 mm particles moisture content is 35%. The liquid limit is 40 - 70%, plastic limit 30 - 50%. However, a special liquid limit of 120% was observed. Clayey mineralogical analysis indicated a small hydrated halloysite content which causes the phenomena of "Thixotropy".

The following properties are based on tests of material under 4 mm grain size: dry density 1.2 - 1.5 ton/m^3 (JIS Compaction) (75 - 94 lb/cu ft); direct shear angle of internal friction 10° - 25°; cohesion 0.3 - 1.0 kg/cm^2 (4.3 - 14 lb/sq in); and coefficient of permeability 1×10^{-7} cm/sec. Tests on material to 50 mm indicate dry density of 1.5 - 1.9 ton/m^3 (Modified AASHO) (94 - 118 lb/ft^3) and permeability of 1×10^{-6} to 1×10^{-7} cm/sec.

It is intended that core will contain 55 - 65% of particles 5 - 150 mm in diameter.

Porosity of compacted soils 35 to 40%, compared to 60% in natural condition. Test fills resulted in dry densities of 1.6 - 1.7 ton/m^3 (100 - 106 lb/ft^3) for sheeps foot roller (27.9 kg/cm^2 pressure), and 1.5 - 1.6 ton/m^3 (94 - 100 lb/ft^3) for pneumatic tire roller. Density did not increase with increased number of passes for this material with 6 - 8% higher than optimum moisture content. Permeability for field density was 4.0×10^{-5} to 2×10^{-6} cm/sec.

Selection of Type

A central core design, Fig. 12(a), was adopted for the following reasons:

a. A central core is considered less susceptible to rupture or cracking due to differential deformation of the embankment, resulting from the steep topography and non-uniform foundation support.

b. An inclined core, if placed at the relatively high moisture content necessary because of prevailing weather conditions, would constitute a zone of relatively low shear strength in the position of a potential slide plane, and thus decrease the stability of the upstream slope.

c. The topography of the site establishes highly desirable locations for the upstream tunnel portals and for the crest of the dam. The space between these two locations is insufficient for a sloping core design.

General Design Considerations

The impervious core has slopes of 10 on 1 to minimize the amount of core material to be placed, since weather conditions suitable for moisture-controlled earth fill operations will be limited to about 100 days per year. During construction, lateral drainage to the outer slopes of the narrow core will prevent the occurrence of excessive pore pressure, even though the material is placed at the upper limit of workable moisture content.

TABLE 5.—MAKIO DAM, DESIGN DATA FOR STABILITY ANALYSIS

Zone and Material	Impervious core	Sandgravel Fill	Rock fill	Sand Foundation
Dry Density in tons per cu m	1.40	2.20	1.65	1.70
Moisture Content in %	30	5	0	22
Wet Weight as Placed	1.82	2.31	1.65	---
Saturated Weight	1.88	2.38	2.04	2.08
Cohesion	0	0	0	0
Value of ϕ, in degrees	31	41.7	45	36
Value of tan ϕ	0.60	0.89	1.00	0.73

Acceleration---0.15g (Acting Horizontally)

TABLE 6.—MAKIO DAM STABILITY ANALYSIS

	P.L.	Pool. Elevation, in meters	Safety Factor
UPSTREAM SLOPE	1	840	1.42
	2	840	1.89
	3	860	1.48
	4	860	1.35
DOWNSTREAM SLOPE	1	880	1.45
	2	880	1.61
	3	880	1.40
	4	880	1.52
	5	880	1.48

Design Data and Analysis

The design data and the results of the stability analysis are presented in Tables 5 and 6. The adopted design constants are based on laboratory tests made by the Aichi Corporation and on tests of similar materials made by the U. S. Bureau of Reclamation, the U. S. Corps of Engineers, and by Electricite de France. The use of comparative results of similar materials was necessary since Aichi Corporation did not have shear test equipment suitable to use with the materials at Makio—all of which have large percentages of coarse materials. Earthquake acceleration of 0.15 g which will act horizontally, is based on probability studies covering 1300 years, and made by Dr. Kawazumi of Tokyo University. "Earthquake tests on models of rockfill dams" were made for Aichi Corporation by Dr. Niwa of Kyoto University. The method of stability analysis used was that of W. Fellenius in, "Calculation of the Stability of Earth Dams". Stability analysis indicated that talus and terrace deposits on the abutments should be removed.

Foundation Treatment

The pattern of intersecting joints, solution channels and faults crossing the site required extensive and careful treatment. The grout curtain, Fig. 12(b), will be carried out by stage grouting. Shafts were excavated in the faulted zones six meters deep and twelve meters long. Gunite shall be used as required. The CO_2 gas in the formation will affect ordinary cement, and a special cement is required. The special cement is based on tests and on specifications used by New York City in 1941.

Embankment Construction

Embankment construction is planned to begin in July, 1959. Core material shall contain not larger than 150 mm size particles; its moisture content shall be limited to that which would permit the satisfactory use of construction equipment, and compaction shall be by sheepsfoot roller in 20 cm loose layers. Transition zone shall contain not greater than 600 mm (23 inch) cobbles and not more than 5% by weight passing a No. 200 sieve, and shall be compacted by pneumatic tire rollers. Filter zone shall contain not greater than 200 mm (8 inch) cobbles and shall be compacted by sheeps foot roller at the time of rolling the transition zone. The upstream rockfill shall be made by end dumping or other means with no requirement for compaction or leveling in regular lifts as long as the fill is brought up reasonably level. The downstream rock fill shall be of quarry run sizes of durable rock dumped and bulldozed into place in lifts approximately 2 meters thick, compacted by controlled movement of the hauling and spreading equipment without sluicing.

Investigation, design and construction supervision has been by Erik Floor and Associates, and personnel engaged on this project includes D. J. Bleifuss, P. T. Bennett, J. L. Schnitz, F. A. Nickell and G. C. Gilboy.

CALVIN V. DAVIS,[3] **F. ASCE.**—Thanks are expressed to Mr. Sakurai for his discussion of this subject. The data concerning the Makio Dam is a significant addition. It was of interest to note that the steep valley cross section and relatively low shear strength of the core material, common to both the Makio and Derbendi Khan dams, dictated the use of a thin vertical core section in both cases rather than a sloping core section.

Because of the similarity between the two projects and considering that construction of the Makio Dam is in an early phase, it is believed to be of interest to discuss the most recent construction experiences at the Derbendi Khan Dam. River closure and diversion through the 6 and 9 m tunnels were made in June, 1958. Since that date the wet season cofferdam has been completed and a total of 2,000,000 cu yd of fill have been placed. Fill operations have lagged behind schedule because of delay in completion of the grout curtain along the center of the cutoff trench. This problem has been remedied by constructing a concrete gallery in the rock along the ungrouted portion of the foundation. Curtain grouting is being completed from the gallery while placing core material over the gallery. Final cleanup of the excavated foundation rock is being made just ahead of fill placement to minimize the length of time which the compaction shales are exposed to desiccation by weathering. This procedure has worked out quite well and will be continued throughout construction.

Although design assumptions have been confirmed in general as construction has proceeded, foundation conditions were discovered to be somewhat more critical than originally determined from the exploration program. Initial grouting operations, including extensive water pressure testing of the grout and check holes, disclosed that the top 10 m of rock were quite leaky. Because of these conditions, the grouting program has been expanded to include consolidation grouting along the dam axis on either side of the main grout curtain and within minor fault zones. Consolidation grouting consists of six rows of holes on either side of the main line of curtain grout holes spaced 1.6 m on centers both ways.

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[3] Pres., Harza Engrg. Co., Chicago, Ill.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3085

ROCKFILL DAMS: DESIGN OF COUGAR CENTRAL CORE DAM

By Paul Thurber,[1] M. ASCE

With Discussion by Messrs. J. B. Cooke; and Paul Thurber

SYNOPSIS

The design of a rockfill embankment for Cougar Dam involved a choice between a central-core and a sloping-core section. The initial decision was for a sloping-core section but this was later reversed and a central-core section was adopted. The principal factor in making the final decision was the uncertainty as to the amount of water load settlement that could be expected in a rockfill embankment the height of the proposed dam and the susceptibility of a sloping core of compacted earth to rupture from the type of settlement that would occur.

INTRODUCTION

Cougar Dam and Reservoir will be a unit in the general comprehensive plan for flood control, navigation, and other purposes in the Willamette River Basin in northwestern Oregon which was authorized by Congress in the Flood Control Act of 1938 (Public Law 761, 75th Congress, 3rd Session). The primary function of the project will be flood control. Other functions will include power generation and storing water for release during periods of low natural flow to aid irrigation, navigation, and pollution abatement on the lower stretches of the river system.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1749. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Chf., Foundations and Materials Branch, U. S. Army Engr. Dist., Portland, Ore.

The dam site is located on the South Fork of the McKenzie River, 4.4 miles above the confluence of that stream with the McKenzie River and 42 air miles east of Eugene, Oregon. (Figs. 1 and 2) The dam, as now planned, will be 1580 feet long at the crest and 445 feet high above the foundation with the axis arched upstream. It will control the runoff from 210 square miles of mountainous and timbered land on the western slopes of the Cascade Mountains, practically all of which is within the Willamette National Forest. The reservoir will cover 1280 acres at maximum pool and will have a storage capacity of 219,000 acre-feet, of which 155,000 acre-feet will be available for flood control storage.

Appurtenant structures will include a concrete spillway on the right abutment, power and regulating outlet tunnels through the left abutment with a common intake structure, and a powerhouse near the downstream toe of the dam. The spillway discharge will be controlled by two tainter gates, each 40 feet wide by 43.5 feet high. The power tunnel will be steel-lined, 10.5 feet in diameter and 1200 feet long, and the regulating outlet tunnel will be concrete-lined, 13.5 feet in diameter and 1000 feet long. The initial power installation will be two 12,500 kilowatt units with provision for expansion to accommodate an additional unit if future power demand justifies a peaking plant.

The diversion tunnel, a horseshoe-shaped unlined tunnel 19.5 feet in diameter and 1800 feet long, was completed in 1957 and approximately 1,400,000 cubic yards of talus overburden was removed from the abutment slopes under a separate stripping contract during that year. A road along the west side of the reservoir, to replace the existing Forest Service road in the valley, is under construction and will be completed in the late fall of 1958. It is planned to award the contract for construction of the dam and appurtenant structures in the early spring of 1959 with completion scheduled for the fall of 1961.

Geology and Foundation Conditions

The dam site is located in the Western Cascades, a geologic province of early and middle Tertiary pyroclastics, lava flows, and contemporaneous intrusives. The oldest rock in the reservoir area is a series of bedded pyroclastics which form the valley floor at the dam site. Following the accumulation of these pyroclastics there was a gentle regional warping which tilted the area toward the east. After several cycles of vigorous erosion followed by renewed volcanic activity which filled the eroded valleys with lava flows and additional pyroclastic deposits, the formation of the present Cascade mountain range blocked the drainage to the east, resulting in extensive valley filling with alluvial material and starting the drainage pattern to the west which exists today.

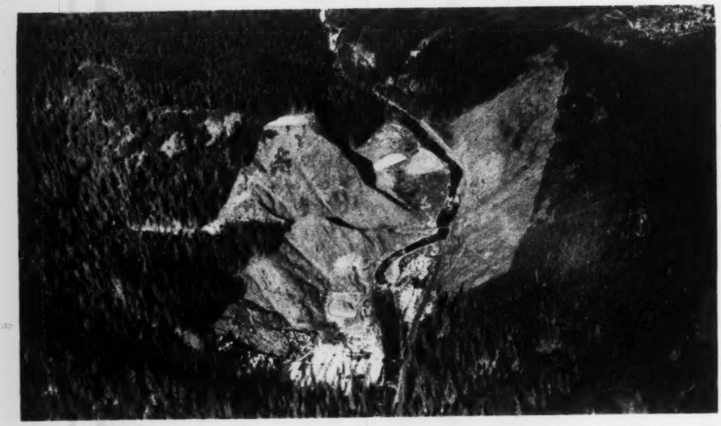


FIG. 1.—VIEW OF DAMSITE LOOKING DOWNSTREAM

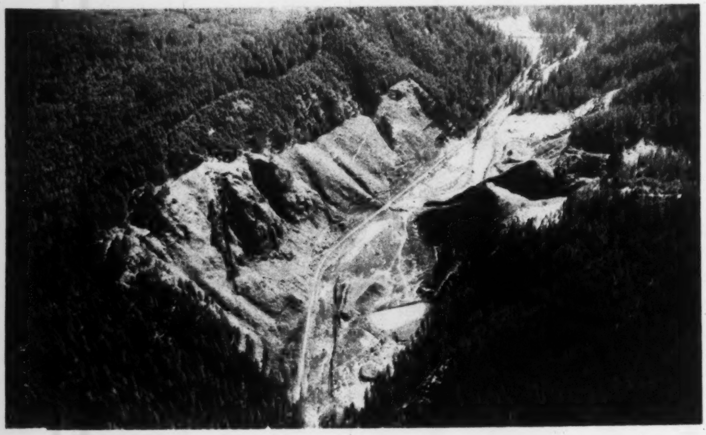


FIG. 2.—VIEW OF DAMSITE LOOKING UPSTREAM

The region is characterized by prominent sharp ridges and steep heavily timbered slopes with shear rock cliffs and broad high-level benches, which form a topography of early maturity. The valley slopes are mantled with a mixture of residual soils, slope wash and talus, and the valley floors are covered with varying depths of recent river alluvium. The river flows in a bouldery channel with many rapids and constrictions produced by boulder terrace deposits and by irregular erosion of the pyroclastic rocks and igneous intrusives. At the dam site it has cut a precipitous canyon some 700 feet deep through a basalt mass.

A geologic section of the dam site at the axis is shown in Fig. 3. The bedrock in the valley floor is a flat-lying series of alternating layers of mudstone and tuff. The results of pressure tests in core drill holes in this formation indicates that foundation leakage will not be a problem, and it is planned to limit the initial grouting in the valley floor to exploration grouting and to expand this program in areas where the grout take indicates that additional grouting is necessary. The overburden on the valley floor is a mixture of sand, gravel and boulders which varies in depth from 5 feet to 30 feet except in potholes where a depth of 70 feet has been measured. This alluvium, except for a shallow surface mantle of sand and silt, has been well consolidated at all locations where it has been explored. The surface mantle will be removed from the entire area under the embankment, but it is not planned to excavate to bedrock except under the core and possibly in an exploration trench at the downstream toe of the dam.

The sides of the canyon are formed by massive basalt outcrops which overlie the tuffaceous sediments in the valley floor and rise almost vertically from the valley slopes to heights above the crest of the dam. These basalt masses are backed by a series of tuffaceous sediments similar to those in the valley floor which outcrop above the basalt on both abutments. The contact of the basalt and the tuffs was tight and usually welded in all the drill holes in which it was encountered. The overburden on both abutments is a mixture of basalt talus and sandy silt slope wash. The major accumulation of this talus overburden is downstream from the axis on the left abutment and depths of 100 feet have been measured in drill holes in this area. All of this material will be removed from the foundation area under the embankment.

The principal joint pattern in the area consists of two nearly complementary sets of joints dipping from about 70 degrees to vertical. These joints occur in the basalt on both sides of the canyon. On the left side one set of joints is nearly parallel to the canyon wall and the wall downstream from the axis is formed by almost vertical slabs more or less separated along these joints. (Fig. 4) The rock exposed above the talus slope is sound basalt, but when the talus was removed it was found that the basalt which had been covered was badly weathered and disintegrated to depths of about 30 feet. After the talus and weathered rock was removed from in front of the vertical slabs, some sections toppled into the excavation indicating that the bases of these slabs are also disintegrated. It is believed that the disintegration resulted from the action of water trapped in the open joints and in the talus. This condition is limited to the area downstream from the axis on the left abutment (Fig. 4) but will require the extensive removal of weathered rock and the removal of some sound rock in that area. It is anticipated that more extensive grouting will be required in the abutments than in the valley floor because of the jointing in the basalt. The extent of the proposed grout curtain is shown in Fig. 3. Horizontal holes will be drilled into the left abutment along the slope of the

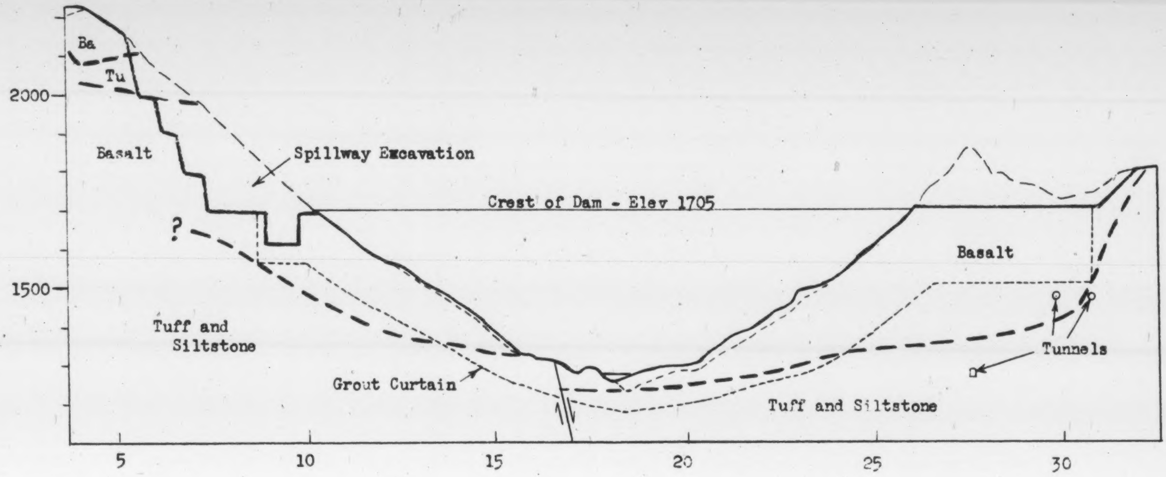

2000
1500
Ba
Tu
Basalt
Spillway Excavation
Crest of Dam - Elev 1705
?
Tuff and Siltstone
Grout Curtain
Basalt
Tunnels
Tuff and Siltstone
5
10
15
20
25
30

FIG. 3.—GEOLOGIC SECTION AT AXIS

FIG. 4.—VIEW OF LEFT ABUTMENT

embankment downstream from the axis to provide drainage and prevent the accumulation of water in the open joints in the basalt.

There are no known records of seismic activity originating in the vicinity of the dam site, but faulting is a prominent feature of the regional picture and some faults of appreciable offset cross through the site. These have not been considered in the embankment design because they do not appear to have suffered movement in recent geologic periods and those exposed in excavating the diversion tunnel were tight and well healed.

Embankment Materials

Rock for the embankment will be obtained from required excavation for the spillway and other structures and from quarries in the basalt outcrops which form the sides of the canyon at the dam site. The principal quarry will be developed in the rock face downstream from the spillway on the right abutment. The basalt exposed in this face extends from approximate elevation 1400 to approximate elevation 2100 and about 1800 feet downstream from the limits of the spillway tailrace channel. Jointing varies from 4 to 12 inches in the upper portion of this exposure, from 6 to 36 inches in the central portion and from 18 to 60 inches in the portion below approximate elevation 1500. A secondary quarry can be developed in the area upstream from the embankment on the left abutment. The rock is a good quality basalt, weighing 150 to 160 pounds per cubic foot, with a compressive strength above 20,000 pounds per square inch. Weathering and regional tectonics have opened the joints to varying degrees and the rock sizes which can be obtained will be largely controlled by the extent to which this has occurred. Weathering, except at shear planes and faults, is generally limited to an oxidized film or thin coating along the joints which does not impair the quality of the rock.

Locating a source of material for the impervious core in the vicinity of the dam site has been a problem. The larger deposits of suitable material occur as terraces on the upper portion of the valley slopes. Several of these were investigated with auger borings and bulldozer trenches and all were found to contain similar material. The most favorably located sources are two areas on the slopes above the right abutment, about 1.5 miles from the site by haul road. The material is a sandy silt with a liquid limit ranging from 45 to 60 and a plastic index ranging from 11 to 17. The natural moisture contents in all deposits explored were well above the optimum and the material will have to be dried before it can be compacted with the heavy pneumatic-tired rollers which will be used. The shear strength is high when the material is compacted at the moisture contents at which it can be used. Several deposits of gravelly material were explored in an effort to find a material suitable for use in the core which would be easier to work and could be worked over a longer season. The materials in the deposits explored contained sufficient fines to be impervious but they also contained cobbles and boulders in such amounts that processing would be necessary to remove them and this would be difficult because of the high moisture content and the clayey nature of the fines.

Approximately 1,400,000 cubic yards of material have been removed from the abutment slopes and disposed of in a waste area just upstream from the dam site. This material is a mixture of basalt talus and sandy silt, with varying amounts and sizes of talus rock. Much of this material appeared to be suitable for use in the core but some portions did not contain sufficient fines and all contained large rock and boulders. An effort was made to segregate the more suitable material in the disposal area and dragline trenches are being excavated so that this material may be examined more carefully and samples obtained for laboratory testing.

Material for the transition zone will be obtained from river bars and adjacent flood plain deposits in the vicinity of the dam site but it will probably be necessary to manufacture the filter materials in order to obtain the required gradation. It is anticipated that filter materials and concrete aggregates will be manufactured from the local gravels in a combined operation since no satisfactory natural aggregates have been found.

Embankment Design

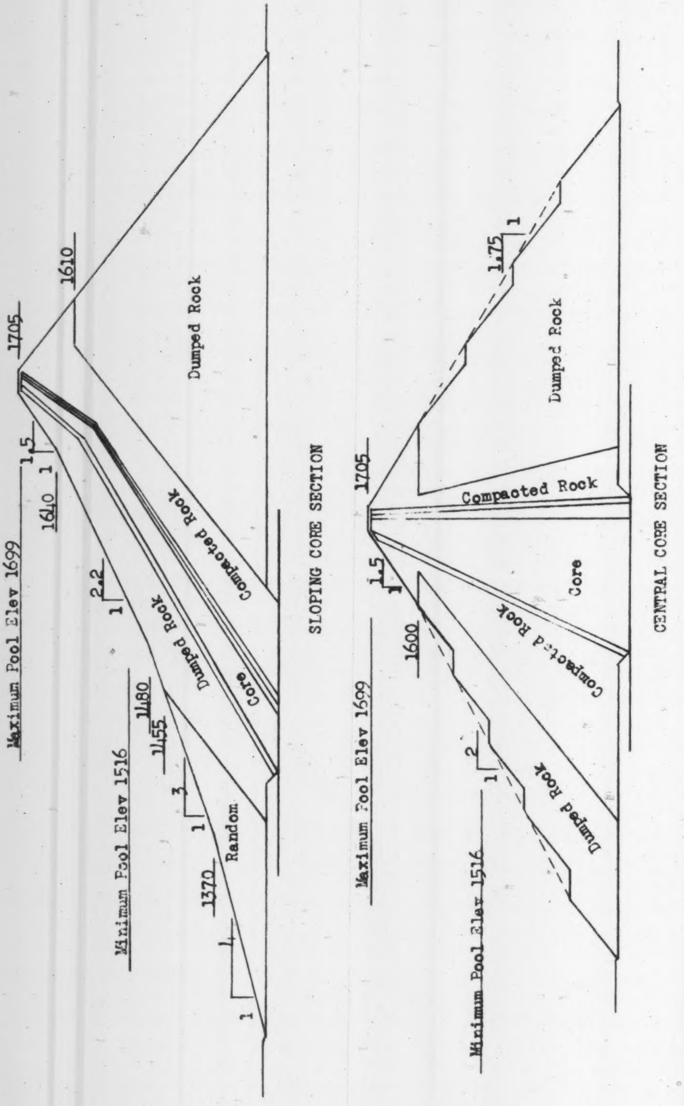
The design originally proposed for Cougar Dam in House Document 531, 81st Congress, 2nd Session (1950), was a gravel embankment with a compacted earth core, similar to the designs used for other dams in the Willamette River Basin. It was found, however, early in the advance planning investigations, that the construction of this type of embankment would require extensive borrow pits in gravel deposits near the mouth of the South Fork and that local interests would object to this excavation because it would impair the natural beauty of the area. Since an adequate quantity of rock for a rockfill embankment could be obtained from the basalt outcrops which form the canyon walls at the dam site, studies were made of the comparative costs of a gravel and a rockfill embankment. The results indicated that the total project cost would be less with a rockfill embankment than with the gravel embankment originally proposed.

Two types of rockfill embankment were considered, one with a centrally located compacted earth core, the other with a sloping compacted earth core located upstream from the axis. The central-core design would require less foundation excavation and a smaller quantity of impervious core material but the sloping-core design has the advantage that, under normal weather conditions, it could be constructed in a shorter period of time with a consequent saving in construction cost. The long rainy season in the Willamette River Basin limits the period during which impervious materials can be placed to from three to four months in normal years and may limit it to less than two months in wet years. Rock, on the other hand, can often be quarried and placed the year round at the elevation of the dam site, and for all except two or three months during the most severe winters. With a central-core design the rate at which the rockfill can be constructed will be controlled by the rate at which the core material can be placed. On gravel embankments constructed in this area it has usually been possible to continue placing gravel after the fall rains have halted the placing of core material but the length of time during which this can be done is limited, particularly during the later stages of construction, by the height to which the gravel can be carried above the core. When this height is reached it is necessary to stop all work on the embankment until the placing of core material can be resumed the following summer. With the sloping-core design the rockfill section downstream from the core, which contains the major portion of the rock in this design, can be completed independently of the core, and scheduling studies indicate that this could decrease the time required for construction by as much as one year.

The sloping core section shown in Fig. 5 was adopted because the results of the preliminary studies indicated that a dam of this design could be constructed at a substantially lower cost than one with a central core. It was planned to place the rock in the downstream dumped-rock section in one lift by dumping and sluicing from elevation 1610 and to face this with a compacted rock section to support the core. The compacted rock would be placed in 12-inch to 18-inch lifts and compacted with a 50-ton roller equipped with steel wheels.

Since the proposed dam will be appreciably higher than any rockfill dam with a sloping core previously constructed, 445 feet above the foundation compared with 325 feet for Kenney Dam, there was considerable concern about the amount of settlement that would occur when the reservoir was filled and the possible effect of this settlement on the sloping core. Detailed information was obtained on the settlement of three rockfill dams with reinforced concrete faces, Salt Springs Dam (328 feet high), and the Lower Bear River Dams (245 feet high and 150 feet high) constructed by the Pacific Gas and Electric Company in California. Essentially all of the water load settlement in these dams took place during the first complete filling of the reservoir. Two types of movement were observed in each dam, a settlement normal to the concrete facing which was maximum in the central portion of the dam about four-tenths of the height above the base, and a lateral settlement away from the abutments which was maximum at the abutments and tended to open joints and cause cracks in the upper portion of the concrete facing at or near the abutments. The lateral settlement in each case was roughly proportional to, and appeared to result from, the normal settlement.

The settlement data on these dams indicates that the magnitude of water load settlements in rockfill dams may increase at a rate greater than in direct proportion to the height of the dam. Extrapolation from the measured


Maximum Pool Elev 1699
1705
1610
Dumped Rock
1.5
1
1640
Compacted Rock
2.2
1
Minimum Pool Elev 1516
1480
1455
Dumped Rock
Core
3
1
Random
1370
1
1
Maximum Pool Elev 1699
1705
1.5
1
1600
Compacted Rock
Dumped Rock
Core
Compacted Rock
2
1
Dumped Rock
1.75
1
Minimum Pool Elev 1516

SLOPING CORE SECTION

CENTRAL CORE SECTION

FIG. 5

settlements indicates that a maximum settlement of 6 to 10 feet normal to the face might be expected in a similar dam the height of the proposed dam. It is expected that the settlement of a sloping-core rockfill dam will resemble that of a concrete-faced rockfill dam and could be greater because of the flatter slope on which the water load is applied. The probable high maximum settlement normal to the face of the core was not regarded as a problem in itself but the possibility of cracks developing in the core due to lateral settlement of the rock fill was thoroughly studied. Tension in a rock fill, such as occurs near the abutments, appears to open cracks in the rock fill and, in a sloping-core dam, would tend to open cracks in the core. It was observed that cracks normal to the axis opened at Salt Springs dam above the level of the partially filled reservoir during the first year. Arching the core in plan would not protect it if cracks developed ahead of the rising reservoir level because water would enter the cracks before there was sufficient water pressure to close them by arch action.

The problem of protecting the core against possible rupture became more serious as the design progressed. The relatively steep abutment slopes at the proposed site will be conducive to large lateral settlements in a rockfill embankment and the topography at the left abutment is such that it would be difficult to avoid high tensile and shearing stresses in a sloping core. The most favorable location for the axis at the left abutment is along the crest of a steep-sided basalt ridge which projects into the valley at this point. In a sloping-core design the base of the core would rest on the upstream slope of this ridge and the core would be supported by a varying depth of rock fill with an abrupt change in depth at the crest of the ridge. It was agreed that compacting the rock fill which supported the core would afford some protection to the core, and the possibility of compacting the entire downstream rockfill section was considered, but it was questioned if there could be any positive defense against cracking of a sloping core in a dam of the proposed height at this location. The suitability of the available core material for a sloping core was also questioned because laboratory tests indicate that it is an MH material which would tend to crack rather than deform under tensile stress and would be very susceptible to erosion. Since the original decision in favor of the sloping core design had been based primarily on a comparison of estimated costs of the two designs it was decided to restudy this comparison. The results of these studies indicated that, if the sloping-core design were modified to meet the objections, it would no longer have any advantage over the central-core design in either cost or time required for construction, and there would still be a question if it were the preferable design for the proposed location. This was the basis for reversing the original decision and adopting the central-core section shown in Fig. 5. The plan for this section is shown in Fig. 6.

The dumped-rock sections will be constructed of clean, sound basalt in quarry-run sizes with a limitation on the amount of fines, placed in lifts not more than 100 feet in height by dumping and sluicing. The contractor will be permitted to vary the lift heights to fit his construction program. The outer slope of each lift will be the natural dumped-rock slope and the required average slope will be obtained by berms at each lift. The downstream compacted rock section and the upstream compacted rock section above elevation 1600 will be constructed of similar rock with a 12-inch maximum size, placed in shallow lifts and compacted. The lift thickness and amount of compaction to be required will be determined by test fills to be constructed this summer.

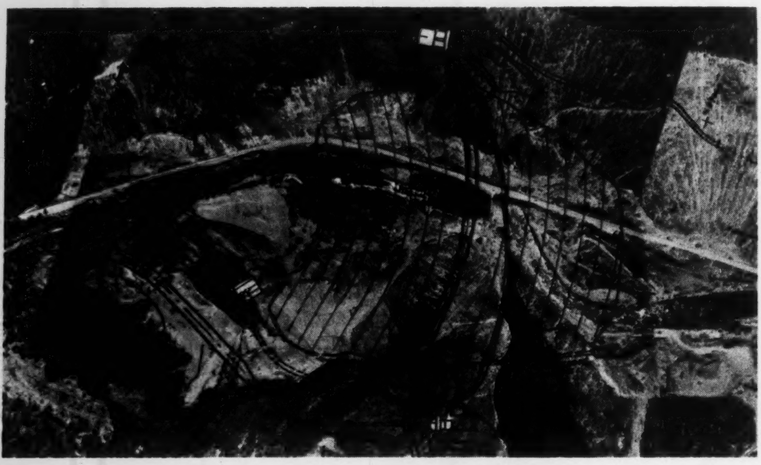


FIG. 6.—PLAN OF COUGAR DAM

The quality requirements for the rock placed in the upstream compacted rock section below elevation 1600 will be relaxed to permit the use of partially weathered basalt, sound tuffs obtained from required excavation, boulder alluvium excavated from the core trench, and the cleaner talus obtained from abutment stripping. Placing and compacting requirements will be similar to those for the other compacted rock sections. The downstream compacted rock section shown is a minimum section, and it is not expected that the downstream boundary will be exactly as shown. When the dumped-rock section is constructed in advance of the core section the compacted rock will be placed against the dumped rock slopes, and when the core section is constructed in advance of the dumped-rock section the compacted rock section will be of the thickness required to support the core.

The design of the core section has not been completed at this time. Present thinking is that it will be a two-section core, with material obtained from borrow areas in the upstream section and material obtained from abutment stripping in the downstream section. The final decision on this will not be made until investigations now being made of the materials stripped from the abutments are completed. The upstream slope of the core will be protected by a transition zone of gravel and sand and the downstream slope by a two-zone filter. A zone of rock spalls may be used downstream from the filter, depending on the results obtained in the compacted rock test fills.

The estimated total in-place volume of the embankment now being designed is 13,200,000 cubic yards, which includes 11,000,000 cubic yards of rock, 1,600,000 cubic yards of core material, and 600,000 cubic yards of filter and transition zone material.

ACKNOWLEDGMENTS

The Board of Engineering Consultants for Cougar Dam is composed of Arthur Casagrande, M. ASCE; James P. Growdon, M. ASCE; and I. C. Steele, M. ASCE. The dam is being designed by the U. S. Army Engineer District, Portland, Oregon; Colonel Jackson Graham, District Engineer; Ben L. Peterson, Chief, Engineering Division. The writer is grateful to Mr. Peterson for his review and constructive criticism, and acknowledges his debt to Robert J. Pope, Chief, Soils Engineering Section, who supervised the embankment design, and to Lloyd L. Ruff, Chief, Geology Section, who supervised the explorations and foundation design, for their contributions to this paper.

DISCUSSION

J. B. COOKE,[2] **F. ASCE.**—Mr. Thurber has presented a concise and valuable paper on the engineering considerations of geology, foundation, materials and design for a rockfill dam of unprecedented height. The thoroughness and caution in the considerations is particularly appropriate since the unprecedented height is associated with less than ideal geology and materials. The use of experience data and measurements on previous dams, mentioned by the author, stresses the value and importance of this Symposium in making such information available to all engineers engaged on planning and design of dams. The experience information is increasingly important in the selection of type and design of rockfill dams of still greater height. The availability of considerable experience data made possible through this Symposium in no way reduces the need of the pooled judgment and experience of many engineers on any specific project, such as recognized in Mr. Thurber's acknowledgments.

Regarding streambed excavation, the author states, ". . . not planned to excavate to bedrock except under the core and possibly in an exploration trench at the downstream toe of the dam". Where it is reasonably feasible to place the core on bedrock, it is done. Where it is not, designs with an upstream impervious apron, such as for Cheakamus Dam and the proposed High Aswan Dam, can adapt the rockfill dam to the site. The leaving of substantial amounts of streambed materials in the foundation of a rockfill dam is often of major economic importance and in most cases may result in a better dam. However, it is considered that a properly backfilled exploration trench at the downstream toe is a necessary part of a rockfill dam intended to be founded on streambed materials.

The careful considerations discussed under the heading "Embankment Design" emphasize the value and importance of measurements of lateral movements of points on the crest of a rockfill dam. The term lateral movement refers to the "across canyon" component, which is parallel to the axis of the dam. Both the magnitude and the design importance of these movements are considered to increase at a rate greater than in proportion to the height of dam. For some concrete face rockfill dams less than 150 feet in height, the lateral movements have been observed to be negligible. For a well designed and constructed core rockfill dam of less than about 325 feet, these lateral movements are perhaps not significant. However, to be assured that the performance of a rockfill dam is satisfactory, and to provide a basis for improved design of higher dams, it is considered that lateral measurements of points on the crest should be included as a design and operating feature of all rockfill dams of significant height.

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The central core section shown on Fig. 5 of the paper indicates zones of compacted rock both up and downstream from the core. The reasons for the compacted rock sections are not expressly given in the paper. It appears that the compacted rock sections permit the utilization of the smaller quarried rock in the dam and provide some construction advantages in coordinating the core and rockfill construction. The minimum thickness of the downstream zone of compacted rock is probably a basic design requirement as a compacted transition zone. It would be of interest to know the extent to which the compacted rock zones were considered necessary for structural design reasons for this high dam. Also, the results of the test compacted rockfill would be of value. Neglecting the fact that the compacted zone permits the use of rock that might not be suitable for dumped rockfill, how do the bid prices of compacted and dumped rock compare.

At the time of writing the paper, design of the core and rockfill section was not completed. Considerable design progress has been made since, resulting in the award of the construction contract in June 1959. It would be of particular value and interest that the author bring the design history of Cougar Dam up to date.

PAUL THURBER,[3] **M. ASCE.**—The embankment design for Cougar Dam had not been completed at the time the original paper was published in August, 1958. The completed design differs slightly from the design reported in that paper, both in section and in materials. The embankment section is shown in Fig. 7. The modifications, in general, were based on information obtained from additional investigations of the site and available materials. This closure describes those modifications, discusses the reasons for adopting them, and reports on the progress that has been made in construction of the dam.

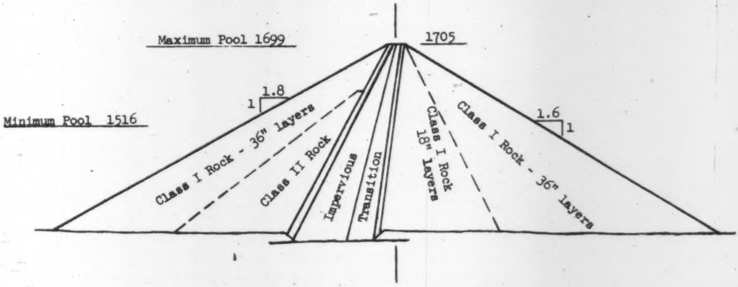

FIG. 7.—EMBANKMENT SECTION

The results, obtained from the investigation of the material stripped from the abutments and stockpiled upstream from the site, indicate that this material is more suitable for use in the impervious core than the MH material originally selected. This material is, predominantly, a mixture of basalt talus and sandy silt, with 25% to 55% passing a No. 4 sieve and 10% to 30% passing a No. 200 sieve. The fine fraction is a border-line ML-CL material. Additional explorations have located a source of suitable transition material in an old terrace downstream from the site. The material is a well-graded gravel and sand with 20% to 45% passing a No. 4 sieve and 0% to 15% passing a No. 200 sieve. The fine fraction ranges from non-plastic thru ML to CL material. The same placing procedures will be used for both materials; boulders and rocks larger than 6 in. will be removed, and the materials will be spread in 12-in. layers and compacted with four coverages with a 50-ton rubber-tired roller.

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[3] Chf., Foundations and Materials Branch, U. S. Army Engr. Dist., Portland, Ore.

The placing moisture content of both materials will be controlled within a narrow range of optimum.

The base of the impervious core has been moved upstream so the contact with the rock foundation will be upstream from the crest of the steep-sided basalt ridge which projects into the valley at the left abutment. Although the core is inclined, the dam is considered to be a central-core design because the inclination is not sufficient to require flattening the lower portion of the upstream slopes for stability. The impervious core is narrower than in the previous design because the transition material is sufficiently impervious to serve as a secondary core. The transition zone also eliminates the need for the two-zone filter in the previous design. The upstream slope of the impervious core will be protected by a layer of transition material, and there will be a layer of rock spalls between the transition zone and the rockfill downstream.

It was learned, from excavation for the relocated forest service road on the left abutment, that the jointing in the basalt formations is such that large rock suitable for placing in high lifts and sluicing as originally planned will be difficult to obtain; the placing procedure for rockfill has been modified accordingly. Rock from the road excavation which was dumped over the left abutment to be used later in construction of the dam was so broken up by the impact that quarrying methods, which drop the rock more than 20 ft, are prohibited in the contract specifications.

The rockfill zones, shown on the embankment section, are designed to provide a section of well-compacted rockfill to support the core and to use the available materials as effectively as possible. Class I rock will be clean, sound fragments of unaltered basalt or andesite with not more than 12% of the fraction passing the 6-in. sieve passing the 1-in. sieve. The rock placed in 18-in. layers will have a maximum size of 12 in. in shortest dimension, and each layer will be compacted by two coverages with a crawler-type tractor weighing not less than 30 tons and two coverages with a 50-ton rubber-tired roller. The rock placed in 36-in. layers will have a maximum size of 24 in. in shortest dimension with not less than 30%, by weight, in pieces larger than 12 in. in shortest dimension, and each layer will be compacted by two coverages with the tractor. If there is a surplus of small-sized rock it may be placed in this zone in 18-in. layers, but each layer will be compacted by two coverages with the tractor. Oversized rock will be placed on the upstream and downstream faces of the dam without compaction. Class II rock will be basalt, andesite, or coarse-grained tuff with up to 25% of weathered pieces permitted, or sandy boulder alluvium from foundation excavation, with a maximum size of 18 in. in shortest dimension and not more than 25% passing a No. 4 sieve. This material will be spread in 24-in. layers and compacted by two coverages with the tractor.

It is believed that compaction of the rockfill downstream from the core will reduce, or eliminate, movements in the dam when the reservoir is filled. This might affect the core, and it is planned to do considerable experimentation with different types of rollers to develop an effective method of achieving this compaction. Plans for a steel impact roller are being developed, and this will be used along with standard rollers of various types.

The contract for construction of Cougar Dam was awarded in June, 1959. The diversion tunnel was constructed under a previous contract, but rock conditions at the upstream entrance required the re-design of the upstream portal, and this structure was included in the dam contract. The contractor was also required to install facilities for trapping salmon downstream from

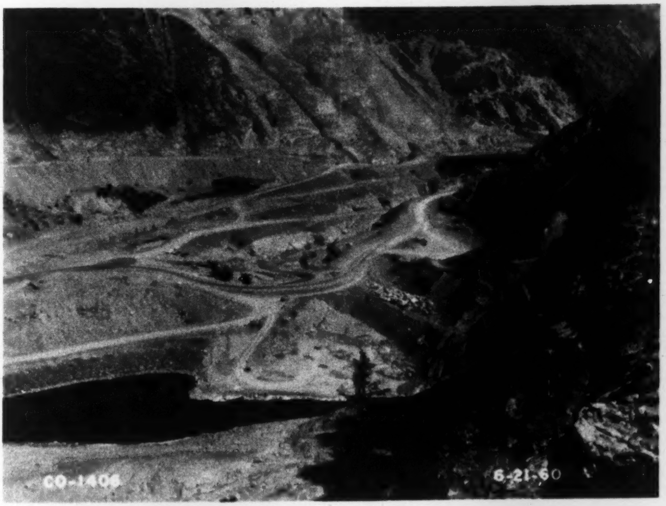
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FIG. 8.—DAMSITE LOOKING UPSTREAM (JUNE 1960) FOUNDATION EXCAVATION IN PROGRESS

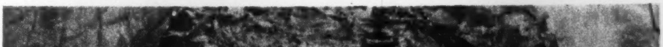

FIG. 9.—PENSTOCK TUNNEL (JUNE 1960)

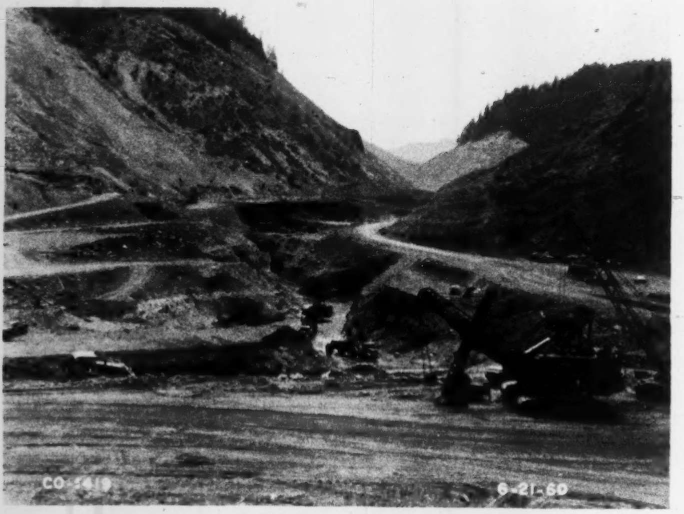
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FIG. 10.—EXCAVATION IN BURIED CHANNEL, LOOKING UPSTREAM FROM DOWNSTREAM TOE

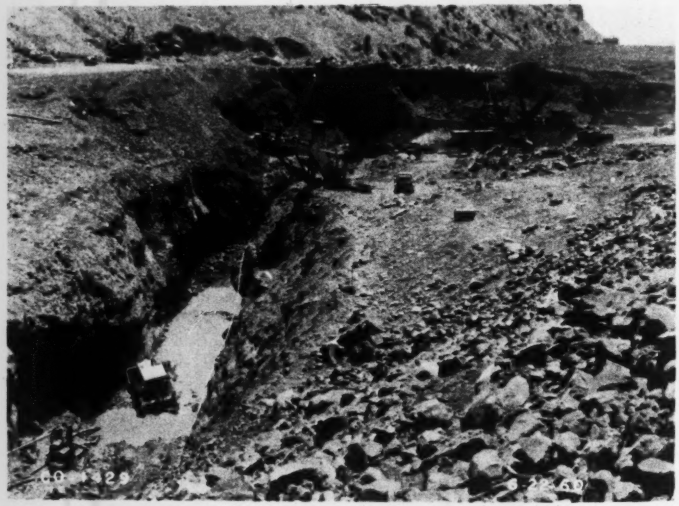
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FIG. 11.—EXCAVATION IN BURIED CHANNEL, LOOKING UPSTREAM THROUGH CORE AREA

the diversion tunnel, transporting them through the reservoir area, and releasing them in the upper river before diverting the river flow through the tunnel. The tunnel portal and fish facilities were completed in time to divert the river flow at the end of the winter flood season in April, 1960. Excavation for the inlet structure and the penstock and regulating tunnels was essentially complete in June, 1960, and considerable progress had been made in excavating the foundation for the dam and in developing the principal quarry on the right abutment downstream from the spillway. Views of the progress that has been made are shown in Figs. 8 and 9.

Exploratory drilling in the valley floor was interpreted as indicating two fairly extensive but unconnected potholes in the foundation bedrock, but excavation in this area has revealed a buried channel that is continuous for the full length of the foundation. This channel is 60 ft to 80 ft wide in the central portion of the foundation where it has been eroded through a basalt cap into the underlying tuff and siltstone, and widens out upstream and downstream where the basalt is not present. The width near the upstream and downstream toes had not been determined when this was written. Views of the excavation in the buried channel are shown in Figs. 10 and 11.

Additional exploratory drilling in the channel has been limited to avoid interfering with the contractor's operations. From information that has been obtained it is expected that the channel will be around 30 ft deep at the upstream toe and 70 ft to 80 ft deep in the core area and near the axis. Present plans are to excavate the material in the channel to rock in the core and to fill the channel with low-strength concrete for the full width of the core, and to excavate the material in the channel to a depth of approximately 35 ft where the channel is less than 100 ft wide upstream and downstream from the core area. The depth of excavation in other areas will not be decided until more information has been obtained.

The writer agrees with Mr. Cooke's comments on the value and importance of measurements of lateral movements of points on the crest of a rockfill dam. His organization has been a leader in obtaining these data, and the designers of Cougar Dam are indebted to them for information made available during the design.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3086

ROCKFILL DAMS: REVIEW AND STATISTICS

By John B. Snethlage,[1] F. ASCE, F. W. Scheidenhelm,[2]
and Arthur N. Vanderlip,[3] F. ASCE

With Discussion by Messrs. F. L. Lawton; G. Robert Koch; and John B. Snethlage, F. W. Scheidenhelm, and Arthur N. Vanderlip

SYNOPSIS

This paper reviews present practices. It develops a definition, classification and terminology for such dams and for those using earth in addition to rock fill. It discusses the advantages of deck-type dams; also settlement and economy of rock-fill dams. In its appendix it lists rock-fill dams constructed in the United States and in Foreign countries.

Unlike most, if not all, of the other papers forming parts of this symposium on rock-fill dams, the authors treat herein the broader aspects of dam construction of that type.

Constructed as expedients during pioneer days in the West, several of those early, though smaller, rock-fill dams remain as functioning monuments to ingenuity and engineering enterprise. Since the close of World War II, rock-fill dams have increasingly received attention. Their importance is attested by the symposium of which this paper is a part.

Note.—Published essentially as printed here, in August, 1958, in the Journal of the Power Division, as Proceedings Paper 1739. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Cons. Engr., East Chatham, N. Y.

[2] Cons. Engr., New York, N. Y. (Deceased).

[3] Cons. Engr., New York, N. Y.

There is of course ample justification for this interest in rock-fill dams. Primarily it lies in the resultant economy, but not in that alone. It is fair to say that various important dams constructed since that war would not exist were it not that selection of the rock-fill type made the construction economically feasible.

In former years a natural and perhaps laudable conservatism on the part of dam designers, seemingly justified by some failures, caused them to shrink from investing millions in large dams consisting mainly of what could be called a heap of loose rock, while in many instances a dam constructed of concrete could be had for nearly the same cost. Perhaps with some persons a like feeling still persists.

Chiefly responsible for the present-day favorable cost aspect of rock-fill dams is the fact that this type of construction is so well adapted to mechanization. Large-capacity quarry shovels and dump trucks, for loading and transporting the loose rock, permit construction rapidly, at a relatively low unit cost and with a minimum of personnel.

Also there is the fact that utilization of Nature's construction material, where plentifully available at or near the damsites, creates a situation well-nigh ideal for damsites which are not easily accessible. That is true, not only in the United States, where most of the readily accessible and otherwise desirable damsites are already in use, but also, as to dam-building in foreign countries. Thus over the last 10 years more high rock-fill dams have been constructed abroad than in our country.

A. Proposed Definition of Rock-Fill Dam

The name "rock-fill dam" is commonly applied to any dam in which rock fill is used. However, that name has been applied to various dams which, although utilizing rock fill, could more justifiably be designated otherwise, e.g., as earth dams or rubble masonry dams.

The term, "rock-fill dam," necessarily implies the use of two different materials, each having its own function. That is, a structure of rock fill alone will constitute an obstruction, but not a complete barrier, to stream flow. It can not truly constitute a dam.

An unsuccessful attempt to make rock fill alone constitute a dam occurred in the case of the Littlefield dam, on the Virgin River, Arizona, where natural silting was relied upon to seal the interstices of a rock fill and hence to provide impermeability. However, when construction had advanced to a height of 126 ft., or 11 ft. below the ultimate crest, a flood washed out the rock fill, well before the stage of overtopping had been reached (Western Construction News, Nov. 29, 1929, p. 618). In short, for a dam of rock fill, there is needed a second material, in order to make the structure impervious.

As yet no adequate definition of a rock-fill dam appears to exist, but the following is a suggestion believed to be worthy of development:

A rockfill dam is considered to be one that relies on dumped rock as a major structural element. Included are rockfill dams of the types with impervious face membranes, sloping earth cores, thin central cores, and thick central cores.

Criteria

It is proposed that the criteria for formulating an appropriate definition of a rock-fill dam be three-fold:

1. The material comprising the fill is loose rock, mainly taken at random, whether from a quarry or other source.

   Although the material is usually derived from quarrying operations, it is not the intention to exclude the use of natural rock slides or talus or rock from excavations for structures. In short, it is proposed that, to qualify as rock fill, the material consist of pieces of rock of various mixed sizes, obtained from whatever source is most convenient and economical.

2. The material is placed in the dam largely or entirely by dumping, regardless of whether the layers of such placing (usually referred to as "lifts") are relatively thin or thick and regardless of method of compaction—mechanical, sluicing or otherwise.

   This criterion would exclude dams constructed in the main of individually-placed rock. Such latter dams are actually rubble or dry-masonry dams and should not be called rock-fill dams.

3. The rock fill constitutes the major portion, i.e., at least 50%, of the maximum cross-section of the dam.

   This would exclude dams in which some other material, usually impervious (and usually rolled) earth, occupies more than half of the cross-sectional area. By "cross-sectional area" there is here meant the area of the maximum cross-section of the dam above a horizontal, or nearly horizontal, line which intersects the upstream and downstream slopes of the dam and which in general separates the man-made structure from Nature's materials upon which the dam is founded.

   For the purpose in point, material below such line, of whatever kind, would not be considered to be part of the pertinent cross-sectional area.

   Dams with very wide central earth cores, occupying more than half of the cross-sectional area, should be considered in reality to be earth dams, even though the material on each side of the earth core be rock fill.

   Likewise, dams sometimes referred to as being of a composite or combination type, that is, with an upstream section of rolled earth and a downstream section of rock fill, would be designated "rock-fill dams" only if the earth section were less in area than the rock-fill section.

   It is of course true that the proposed 50% dividing line is somewhat arbitrary and that rock-fill problems are just as real even though the rock fill may occupy less than 50% of the cross-sectional area. However, particularly for the purpose of distinguishing between earth-fill and rock-fill dams, the selection of the 50% point would be logical.

Proposed Definition

Based on the foregoing, the authors propose the following definition of a rock-fill dam:

> A rock-fill dam is one of which at least half the material in the maximum cross-section is comprised of loose rock placed by dumping.

In Section C hereof, there are illustrations of the application of such definition to various forms of 'fill' dams which utilize rock fill as a primary structural element.

B. Basic Types of Rock-Fill Dams

Rock-fill dams must be provided with an impervious portion or diaphragm —not necessarily a relatively thin one. Consequently the designation of basic type of rock-fill dam may logically be in terms of the location of the diaphragm in relation to the body of the dam. Thus there are two basic types, viz., the deck type and the central-core type. Of these, the former is believed to be the older.

1. Deck Type of Rock-Fill Dams

The general characteristic of this type of dam is that the impervious diaphragm, regardless of material used, is a deck on the upstream side of the body of the rock fill. The deck is relatively thin except generally when it is of earth without an upstream covering of rock.

Although the very highest rock-fill dams in the world are of the central-core type using earth, those of the deck type, in some cases nearly as high, are more numerous.

Deck Material

Various materials have been used in the construction of the impervious decks of rock-fill dams, with refinement to suit the taste of the designers or to follow contemporary habit and experience elsewhere.

It is not within the scope of this paper to discuss the details of construction of such decks; that has been done in other papers constituting parts of the symposium. A mere summary will suffice:

Wood

Planks are nailed onto wooden beams embedded in the upstream surface of the rock fill, which surface generally has been smoothed with a coating of portland cement mortar.

There are several dams with such decks of wood. J. D. Galloway in his paper on rock-fill dams (Trans., ASCE, Vol. 104, p. 18) stated that one of the advantages of wooden decks is flexibility. Existing examples are:

Hillside dam, 80 ft., California, 1904
Cucharas dam, 125 ft., Colorado, 1911
Saddle Bag dam, 47 ft., California, 1921
Torron dam, Sweden, 1936

Steel

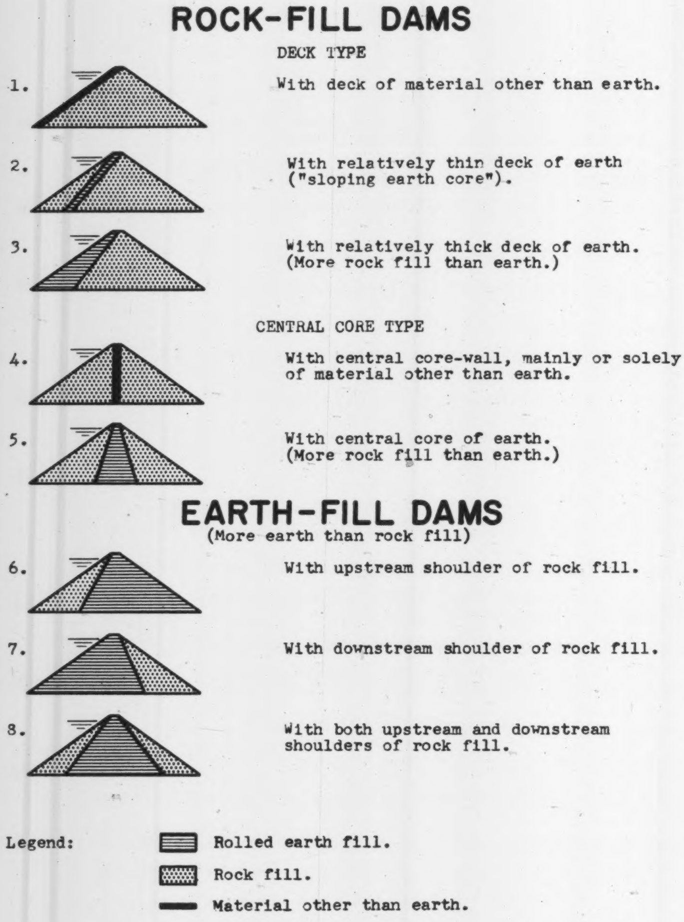
Steel plates, provided with expansion joints, are fastened to steel beams embedded in the surface of the rock fill, similarly smoothed by means of cement mortar.

Details are described in "Steel dams," by O. E. Hovey. Examples of steel-deck dams are:

Skaguay dam, 70 ft., Colorado, 1901
Goose Neck dam, 210 ft., Colorado, 1910
Penrose-Rosemont dam, 100 ft., Colorado, 1932
Pego do Altar dam, 213 ft., Portugal, 1949

Earth (see Section C hereof, illustrative cross-sections 2 and in Fig. 1).

The impervious 'diaphragm' is comprised of earth fill, deposited (with appropriate transition zones) on the upstream side of the rock fill, then rolled



ROCK-FILL DAMS

DECK TYPE

1. With deck of material other than earth.
2. With relatively thin deck of earth ("sloping earth core").
3. With relatively thick deck of earth. (More rock fill than earth.)

CENTRAL CORE TYPE

4. With central core-wall, mainly or solely of material other than earth.
5. With central core of earth. (More rock fill than earth.)

EARTH-FILL DAMS

(More earth than rock fill)

6. With upstream shoulder of rock fill.
7. With downstream shoulder of rock fill.
8. With both upstream and downstream shoulders of rock fill.

Legend:

▤ Rolled earth fill.

░ Rock fill.

▬ Material other than earth.

FIG. 1

for compaction and impermeability. This type utilizes solely Nature's materials, frequently in unaltered form, as found at or near the damsite.

Admittedly this type of dam is ordinarily not thought of as a deck type. However, functionally such use of earth fill is believed to compel classification as a deck type.

This is true even where additional loose rock is deposited over the upstream or outer face of a relatively thin, rolled-earth 'diaphragm,' not merely to protect the earth from wave action but also to ensure its stability. In such dams the use of the presumably more expensive of the two materials is minimized. Dams of this logically evolved sub-type (see Section C, cross-section 2) are frequently referred to as having a "sloping earth core." The principal exponent in North America of this sub-type has been James P. Growdon, M. ASCE.

Examples of dams with this relatively thin form of rolled-earth deck are:

Sillre dam, 26 ft., Sweden, 1933
Nantahala dam, 250 ft., North Carolina, 1942
Kenney dam, 324 ft., Canada, 1952
Brownlee dam, 395 ft., Idaho, under construction

The dams with thicker sections of earth (see Section C, cross-section 3), which have no outer covering of rock (beyond ordinary rip-rap) and which, by reason of the proportions of cross-sectional areas, must be classified as rock-fill dams, are likewise in essence of the deck type. In these dams the upstream slope of the rock fill usually averages steeper than the natural one of dumped rock on the upstream face of the rock fill.

Examples of dams with this relatively thick form of earth deck are:

Ashton dam, 60 ft., Idaho, 1917
Inland dam, 195 ft., Alabama, 1938
Barre Falls dam, 62 ft., Massachusetts, u.c.

In rock-fill dams involving earth decks—as also in the cases of those with central cores of earth—there must be transition or 'filtering' zones in which there is gradation from rock fill to earth.

In this connection it may be noted that similar dams, with the thicker earth fills on the upstream side, have been constructed by the Bureau of Reclamation, but with the downstream portion of the dams comprised of gravel and other pervious material instead of rock fill.

Portland Cement Concrete

The deck material most commonly used has been portland cement (p.c.) concrete. At present, the usual practice is to cast the concrete in panels upon—largely integrally with—a relatively smooth base formed by individually placed pieces of rock over the rough upstream surface of the rock fill proper.

That base is comparatively costly, but serves added purposes, of which the most important and certain is that of providing an upstream slope steeper than the natural slope of dumped rock. This reduces volume as regards both the p.c. concrete and the rock fill itself.

However, there is also a p.c. concrete deck which, apparently without any ill effect, has been placed upon a thick layer of loose, washed gravel, overlying the rock fill proper, namely, in the case of Cogoti dam, Chile (Engineering News-Record, Nov. 5, 1931, p. 725).

In the United States the principal constructor of rock-fill dams with p.c.

concrete decks has been Pacific Gas and Electric Company in California.

Prominent examples of dams with decks of p.c. concrete are:

Dix River dam, 275 ft., Kentucky, 1925
Salt Springs dam, 328 ft., California, 1931
San Gabriel #1 dam, 300 ft., California, 1938
Pinzanes dam, 220 ft., Mexico, 1956
Courtright dam, 310 ft., California, 1958

Asphaltic Concrete

The suitably prepared upstream face of the rock fill is paved with several layers of asphaltic concrete so as to form the impervious deck.

The only examples of dams with decks solely of asphaltic concrete appear to be:

Montgomery dam,[4,5] 113 ft., Colorado, 1957
Smith Mountain dam, 235 ft., Virginia, under construction.

Asphaltic concrete, as a material for the construction of decks for dams comprised of rock, has been used in foreign countries, although not in the form used at Montgomery dam or to be used at the Smith Mountain dam. In no case is the deck known to be solely of asphaltic concrete. Some of such dams are of rubble masonry.

Following are examples of those foreign dams which are known to be true rock-fill dams:[4]

Caritaya dam, 125 ft., Chile, 1935
Genkel dam, 140 ft., Germany, 1952
Henne dam, 200 ft., Germany, 1954
Iril-Emda dam, 246 ft., Algeria, 1954

2. Central-Core Type of Rock-Fill Dams

Central-core, rock-fill dams are those having the impervious portion or diaphragm located at or near the vertical center line of the body of the rock-fill dam. Further subdivision of this type may be made depending on the nature of the core, including its material, e.g., those with relatively thin, central core-walls, generally of some material other than earth, and those with relatively thick central cores, of earth.

Central Core-Walls

The thin, central-core wall is probably a descendent of very old earth dams and dikes, including Dutch dikes constructed several hundred years ago.

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4. "Rockfill Dams: Montgomery Dam with Asphaltic Concrete Deck," by F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip, Transactions, ASCE, Vol. 125, Part II, 1960.
5. "Laboratory Investigation of Asphaltic Concrete—Montgomery Dam, Colorado," by F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip, Presented at the February, 1958, Annual Meeting of the Assoc. of Asphalt Paving Technologists, in Montreal, Canada.

At a time when compaction of earth was accomplished as well as possible, but, as compared with present day means, in a primitive manner, the controlling thought was that a dam or dike constructed of earth alone could not surely be made watertight and that the mere insertion, vertically, of a thin wall of impervious material, approximately along the center line, would make it so.

Because earth slopes as flat as 1 (vertical) on 4 or 5 (horizontal) were then customary, many such dams were successful. However, when economy demanded steeper slopes, now and then some bitter experience resulted.

The use of thin, central core-walls in earth dams of whatever height has practically disappeared and their use for rock-fill dams is becoming very rare, even for relatively low dams. In both cases, the basic reason may be that the material of the fill upstream from the thin, central core is completely submerged and in any case contributes nothing to the stability of the dam. In fact, it adds to the horizontal water pressure against the core-wall and the downstream portion of the fill and thereby reduces the margin of safety.

In such central core-wall, rock-fill dams as exist, the material for the core-walls has been reinforced concrete, plate steel and wood or combinations thereof. None of these materials combines strength, flexibility and durability to a degree sufficient to warrant burial inaccessibly in the interior of a dam.

Existing examples of central core-wall, rock-fill dams are:

East Canyon dam, 93 ft., Utah, 1899
Crane Valley dam, 145 ft., California, 1910
Smith dam, 43 ft., Colorado, 1916
Silverwood dam, 80 ft., Australia, 1928.

Central Cores of Earth

A major improvement, particularly for high dams, is the use of a thick, rolled-earth, central core. It is of course still true that the completely submerged upstream rock fill adds to the horizontal water load against the body of the dam proper (core plus downstream rock fill). However, the thick earth core, together with adjacent rock fill, lends itself more readily to stability computations, approximating those usually made for conventional gravity-type dams of p.c. concrete. The rock-fill sections confine the steep earth slopes (with their appropriate intervening zones of transition from coarse rock to earth).

The highest rock-fill dams are of such construction. Examples of rock-fill dams with central cores of rolled-earth are:

Mud Mountain dam, 425 ft., Washington, 1948
Ambuklao dam, 420 ft., Philippines, 1955
Derbendi Khan dam, 410 ft., Iraq, u.c.
Trinity dam, 537 ft., California, u.c.

Hydraulically-placed or sluiced-earth fill, instead of rolled-earth fill, has been used for the construction of the impervious interior of at least two dams, which, because of the 'shoulder' of rock on each side of the hydraulic fill, have the appearance of rock-fill dams. These are the El Capitan (California) dam and the Cobble Mountain (Massachusetts) dam. However, under the definition of rock-fill dams, i.e., because of preponderance of earth, each must be classed as an earth-fill dam.

C. Classification and Terminology

There are few structures concerning which there is so much confusion in terminology as is true of rock-fill, particularly those in which earth is used. This is evident in casual references but also in more carefully considered correspondence and in the pertinent engineering press.

In the absence of a cross-sectional sketch of a given dam and of accepted terminology, it may be impossible to determine just what type of dam is involved. Frequently the desire for brevity results in ambiguity or vagueness. Surely such characterization as "a rock fill earth fill" structure lacks the preciseness which is generally typical of an engineer.

The authors believe that the illustrated classification (Fig. 1), with terminology, below may be as helpful to others as it has been to themselves. It is in accord with the definition proposed at the end of Section A hereof and with the descriptions contained in Section B.

Mention and diagramming of dams solely of earth fill are omitted, due to absence of rock fill and as involving no possibility of confusion.

While not insisting that the following represents the optimum, the authors do state that it represents the results of earnest effort in that direction and they propose that it be considered for adoption as an outcome of this symposium.

D. Advantages of Deck-Type of Rock-Fill Dams

The deck type of rock-fill dam has various advantages not possessed by the central-core type. The more important of those advantages may be described briefly as follows:

Greater Margin of Safety

Stability of the deck-type dam as a whole against failure by sliding or by overturning is amply assured, largely because no substantial amount of uplift pressure can be exerted. Particularly is this true of dams with decks of other than earth fill. The factor of safety in these respects is very large. On the other hand, in the case of a dam of the central-core type, none of the rock upstream from the core contributes to safety against sliding or overturning—which, however, is by no means the same as saying that by reason of this fact a rock-fill dam with a central earth-core will not be amply safe.

Greater Tolerance of Leakage

Ordinary leakage through the foundation or the deck can not endanger the dam, for the rock-fill is essentially free draining. Even some overtopping, with flow into the rock fill, is tolerable, though of course not desirable.

Because, with the exception of thick, rolled-earth decks, the area of grouting is at or relatively near the upstream edge of the dam, grouting and the major rock-fill construction can proceed simultaneously, though of course not in the same location. Thorough grouting is a time-consuming process, the extent of which generally can not be fully foreseen. Construction delay can be minimized where grouting and dam construction can go on at the same time. Moreover, because ordinary amounts of leakage through the rock foundation can not endanger the dam and where there is no material economic loss by reason of such leakage, grouting can be somewhat less than perfect.

Rapidity of Construction

Except for the rock immediately underlying the deck, the construction of the main or sole rock fill, as the case may be, is unbroken, contrary to the situation with the central-core type. Once the routine of that construction has been established, the situation lends itself to maximum rate of progress, with correspondingly beneficial effect on cost. Naturally this is true to the greatest extent in the cases of decks comprised of material other than earth—for in those cases the proportion of rock is greatest and there is not the somewhat complicating feature of transitions from rock to earth.

Accessibility of Impervious Diaphragm

Again with the exception of earth decks and their rock coverings, the impervious diaphragm or deck remains accessible for inspection and repair. This is true even as regards the part of the deck remaining permanently under water where a hydroelectric development is involved.

Future Increase in Height of Dam

The deck-type of rock-fill dam is better adapted for future increase in height. The addition of the rock fill requisite for increase in height of course presents no problem of importance. As to the deck, there would be involved merely its extension upward at its original or other slope, with corresponding extension of the cut-off and sealing provisions up the abutments.

On the other hand, the fact that at least the rolled-earth form of the central-core type has some countervailing advantages is evidenced by the fact that the very highest rock-fill dams in existence are of that form.

E. The Settlement Problem

The problems of rock-fill dams stem largely from the fact that rock fills settle and that such settlement may leave some, though by no means unresolvable, question with respect to proper functioning of the impervious diaphragm, of whatever nature.

General Settlement

The loose rock of the fill, compacted initially by the combination of its own weight and the impact of dumping, undergoes further compaction, first essentially vertically, by the superimposed load as the structure gains height, and later horizontally as well as vertically when, upon completion of the dam, the impounded water exerts its pressure. This unavoidable process of natural compaction is generally referred to as settlement. Uncertainty as to such settlement constitutes practically the only rock-fill problem of major importance.

Loose quarried rock has irregular surfaces and edges which come to rest against each other in the process of dumping. The gradual increase in load causes these edges or points of contact to spall or crush and hence there is some re-adjustment within the fill. This process continues until the contacts between the pieces of rock have become sufficiently blunt to transmit the load without further spalling or crushing. The resulting settlement appears to continue, at a gradually diminishing rate, for a few years after completion of the structure. Settlement of this type may be assumed to be proportional to the height of the rock fill above its foundation and to be present throughout the fill.

Surplus fill, sometimes referred to as "camber," and a slight convexity, upstream in plan, are usually provided to compensate for such settlement. Obviously convexity downstream in plan should be avoided because fill settlement or movement in a downstream direction, such as would be due to water pressure, would tend to pull the rock fill apart. Relief dam in California, constructed in 1910, has a convexity downstream in plan; a crack developed in its reinforced p.c. concrete deck, at the center of the curve.

Local Settlement

Probably more serious is what may be referred to as local settlement. When, in the process of dumping, pieces of rock become lodged in such a manner that there are relatively large holes within the fill, even the small adjustments, resulting from spalling of edges or points of contact, may cause partial collapse of such holes and hence additional settlement. The cumulative effect may be substantial. It is not distributed throughout the fill, may be independent of height of fill and sometimes results in subsidence which is strictly local.

Although the existence of excessively large openings within the rock fill is largely a matter of chance, the shape of the pieces of rock has important influence. For rock which is generally of a cubical shape, the chances of occurrence of unduly large openings are materially less than for rock of flat, elongated shape.

Yet Genkel dam, Western Germany, 140 feet high, is reported to be constructed of slate quarry waste, accumulated over many years of operation of several such quarries (Bitumen, December 1953, p. 204); in that instance the material must have been flat and elongated, though possibly not in large pieces.

Means of Minimizing Settlement

Although some settlement of rock fill is unavoidable, means are available to minimize it.

Sluicing

One of the most important means for reducing settlement is to direct jets of water, in large quantities and under relatively high pressure, upon and against the rock as it is being dumped; likewise as to the face of the fill between dumpings. The purpose is to cause the pieces of rock to come to rest more nearly in final position than would otherwise be the case. At the same time, such sluicing moves fine material ("fines"), of, say, 4 in. and smaller, away from the contacts between larger pieces, thus in general resulting in more firm contacts. Sluicing is usually done by means of nozzles of monitors, is very effective and in general can hardly be overdone.

Some text books state that such sluicing is done to wash the fines into the interstices of rock already placed. However, the authors suggest that such is the result rather than the beneficial purpose of sluicing. Actually, under sluicing, these fines can not go elsewhere than into such interstices and any resulting beneficial effect, as concerns later settlement, is at best not important in degree. That is, the sluicing process can not be expected to compact these fines to such extent that the result will be to resist materially, if at all, the settlement of the surrounding rock fill. On the other hand, in the construction of Malpaso rock-fill dam, Peru, 1936, fine material (gravel and sand) was brought separately to the damsite for the sole purpose of being sluiced into interstices (Compressed Air Magazine, July, 1939).

Limitation of Proportion of Fines

Restriction of the permissible amount of fines in rock-fill material is very important. Whereas rock fill may be expected to have voids of the order of 35%, the proportion of fines in the material clearly should be far less; otherwise, by reason of intervening fines, the larger pieces of rock may lack the desired direct contact with each other.

Moreover, the combination of an excess of fines and liberal sluicing may result in the filling of the lower interstices to such extent and with sufficiently fine material as to bar or at least retard drainage within what should be a free draining rock fill. In the case of a deck-type dam, there could thus result the danger that water, draining from foundation rock or abutments or even leaking through the deck, would be ponded upstream from such interior barrier and in turn would cause reverse pressure, against the under side of the deck, at a time when the reservoir is drawn down. In the case of Montgomery dam, with the foregoing in mind, it was prescribed that a limited thickness of dumped rock, under and near the deck, should be free of material of less than 3-inch size. Also positive, i.e., open-joint, pipe drainage was provided along and from the bottom of that zone of screened rock.

Specifications for the body of the rock fill should prescribe the maximum permissible proportion of fines and provide positive means of testing and enforcing compliance. The specifications for the recently completed Montgomery rock-fill dam aimed to limit to 5% the proportion of fines (material less than 4-inch size), but at the maximum tolerated 10%. There was liberal sluicing. Settlement so far has been gratifyingly low.

Methods of quarrying may greatly affect the proportionate product of fines. Random blasting and in some cases the coyote-hole method may result in cheaper quarrying but not necessarily in an appropriate rock product. Blasting techniques worked out experimentally at the specific quarry site and for the kind of rock involved are more likely to minimize the output of fines.

It has been suggested that elimination of an excess of fines remaining in the run-of-quarry output might be accomplished by the use of (dump) trucks with slotted bottoms, thus permitting fines to be shaken out of the truck-loads enroute from quarry to dumping site. However, the authors know of no instance where such method has been carried out.

Supervision

Because of the very nature of a rock fill, the temptation is to assume that all that is necessary, to attain an appropriate ultimate result, is to quarry, haul and dump the rock, together with such sluicing as may be prescribed. However, the rock piled up on the quarry bench is not pre-tested, pre-examined as frequently is earth from borrow pits or pre-graded as are aggregates for concrete. All that is known is that the larger part of the pile is satisfactory and utilizable and that the rest, if used, would contribute toward inferiority.

Clearly, therefore, the price of good rock fill includes adequate supervision and inspection, especially at the quarry bench where loading of trucks takes place and at the dumping and sluicing locations. It need hardly be suggested that such inspection is at least as important during night shifts as during day shifts.

Special Compaction

In a sense, a rock fill, given sufficient time, especially under water load, is self-compacting; yet, if no effort is made to minimize settlement, it may become unduly or, in extreme cases, intolerably large.

In the United States and Canada, rock fills for dams have been placed mainly in relatively high stages or lifts. Although economy of construction enters, it seems fair to say that a principal consideration is the belief that construction in high lifts is conducive to minimizing settlement. Without doubt, there results some compacting effect, due to the longer distance of roll of the larger pieces of rock down the face of a relatively high lift.

Then there are those who seek to accentuate initial compaction by means of impact due to greater height of fall of the rock, e.g., from loaded skips carried by cableway.

In contrast, there are cases (largely European) where, apparently for the attainment of like end, rock-fill placement is in relatively low lifts. Under that method, considerable compaction occurs within the shallow lift by reason of the repeated travel over the surface of the lift or layer by heavily-loaded trucks, transporting rock to the dump face. Given sufficient shallowness of layer, it would seem that vibratory rollers might be effective for the purpose. (On the other hand, the resultant creation of additional fines, due to attrition, is of itself undesirable.)

Various special means for obtaining initial compaction, or at least expediting settlement, result in additional cost which must be weighed against the advantage obtained.

It would be of interest and undoubtedly of substantial help for future rock-fill dam construction if discussions of symposium papers were to contribute pertinent information and views supplementing such as may be contained in the individual papers.

Diaphragm Flexibility—a Requirement

Settlement of rock fill can be minimized but not completely eliminated. One must therefore be reconciled to deal with the effect of such settlement as does take place. Primarily this requires that the impervious diaphragm be without danger of rupture.

Presumably the need for pertinent provision is less in the case of rock-fill dams of the central-core type than in those of the deck type. However, as hereinabove indicated (Section B), at least in the cases of central core-wals (material other than earth), the results have not always been satisfactory.

As to the behavior of central cores of earth, information is by no means extensive or positive—largely because the core is buried and there is no manifestation of behavior unless such be unfavorable, let alone resulting in failure.

As for decks, cracking or other forms of failure due to insufficient flexibility are of record in the cases of materials other than earth. Regarding the performance of decks of earth, particularly relatively thin earth-decks (sloping earth-cores), just as in the case of central cores of earth, little is known in the pertinent respect beyond the fact that there have been no failures.

Decks of portland cement concrete undoubtedly have substantial flexibility where relatively small, jointed panels are used. Possibly the same is true where there are used larger panels which are laminated, i.e., composed of thin slabs. In the cases of decks where comparatively thick slabs of large area are used, question may arise. However, it appears that such construction has adequately met the needs of the cases where used—due perhaps to the fact that the slabs are underlain by what are in effect inclined walls of dry rubble masonry.

As for the newer deck material, asphaltic concrete, experiments and experience are believed to warrant the view that, within reasonable limits of temperature and deformation, that material will adjust itself by flow, without rupture, to maintain contact with its supporting base.

In relation to the minimizing of the effect of settlement, too, it is hoped that discussion of the symposium papers will bring out pertinent experience and facts, as well as views, not contained in papers as submitted.

F. Dams of Greater Height and Economy

The following is submitted by way of specific contribution toward the announced purpose of the symposium.

Greater Height

Dams of a given type and height having been constructed and in operation successfully, it may reasonably be assumed that, under equally favorable conditions, dams of like type and somewhat greater height may be constructed and operated successfully. However, the greater the height for which there is precedent, the more the need for caution in taking the step to still greater height—not overlooking the fact that in general the greater the height of a dam, the greater the potentiality of damage in case of failure.

Always there is the necessity for careful analysis and comparison of the pertinent conditions at the site of and otherwise affecting a proposed dam of unprecedented height with the corresponding conditions involved in what is assumed to be the precedent. In the cases of rock-fill dams, there are, among other such conditions, the hardness and toughness of the available rock, the general shape of the pieces or fragments into which it breaks under blasting and the proportion of fines upon such breaking. This, however, is not to say that in all cases all conditions for the dam of greater height need be as favorable as those for the precedent.

Such step by step increase in structure heights is taking place in the case of the new form of deck-type, rock-fill dam, with its deck solely of asphaltic concrete. Thus, in view of the precedent, the 113-ft. Montgomery dam, in Colorado, the authors have developed plans for such a dam which is to have a height of 235 ft.—still low compared with the highest successful rock-fill dams. Construction is being started on this, the Smith Mountain dam for a hydroelectric development on Roanoke River in Virginia. Meanwhile the authors are involved in the consideration of a like dam to be probably somewhat higher than 300 ft., which too would be in the eastern part of the United States.

Greater Economy

Increasingly, rock-fill dams have become a principal means for making economically feasible projects which otherwise would not be so.

In turn, construction and economy of rock-fill dams have been receiving impetus due to the efficiency and economy of equipment for loading and transporting loose rock. It is in large part the manufacturers of such equipment to whom the engineer must look for further improvement and economies. Because of the large quantities of rock required, even small improvements, reducing unit costs, will be of importance.

The industrial law, almost an axiom, that mass production makes for lower cost applies as well to rock-fill dams. The authors are of the view that the deck-type dam, when utilizing for the deck a material other than earth and hence with the body comprised of rock fill throughout, is likely to be of lower over-all cost than if a considerable part of the cross-section were to consist of another material, such as earth, posing its own problems.

Obviously that view would not be well-founded if the cost of the deck, of material other than earth and therefore comprising an inconsiderable part of the total cross-section, were to be so great as to offset the saving in other respects. However, such appears not to be true, even in the case of a p.c. concrete dam constructed upon a base of individually, and hence rather expensively, placed rock. All the more is it not true of a deck of asphaltic concrete. That material was found to be substantially cheaper than any other for the deck of the Montgomery rock-fill dam. It is believed that this would apply elsewhere.

At Montgomery dam the cost of the asphaltic concrete deck, including all work above or upstream from the rough face of the rock fill, was in 1956-7 less than $14.00 per square yard. The asphaltic deck itself, approx. 12 in. thick, placed in four layers and involving about 27,800 square yards, was constructed in about two months.

Much like the case regarding equipment used for handling rock fill, the great amount of paving required for highways and airfields has encouraged manufacturers to develop asphaltic-concrete, paving equipment which with great efficiency can pave continuously large areas in short time. Adaptation thereof to the placing of asphaltic concrete on a slope has already been accomplished in several cases, though probably not yet in the most efficient form.

Likewise there have been developed mixing plants which, practically entirely on wheels, can readily be transported along highways and set up or removed in a short time; there are large enough for deck construction for rock-fill dams. It is believed that the further adaptation of such paving and mixing equipment will result in still greater economy.

APPENDIX.—LISTING AND STATISTICS OF ROCK-FILL DAMS

Listing and Statistics of Rock-Fill Dams

In the course of their studies of rock-fill dams, the authors have accumulated a mass of information on the subject. Part thereof appears in the tabulations herewith.

These tabulations were intended to list, with certain standard data, all dams which, in accordance with the proposed definition (see Section A of this paper), appear properly to be classed as rock-fill dams. Application of that definition has resulted in elimination of some dams which heretofore have been referred to by others as "rock-fill" dams. Among the more important of the eliminations are some Algerian dams of individually placed rock or rubble masonry and a few others, the central earth-cores of which appear to exceed, as to cross-sectional area, the rock-fill portions of the dams.

There are two tabulations, the first comprising rock-fill dams in the United States, the second those in foreign countries.

Explanatory

The following is in explanation of the tabulations:

Names of dams are those presently used by the owners. In former years some were known by another name. (Notably, Cogswell (formerly San Gabriel No. 2), Goose Neck (Cheeseman), Hillside (South Lake), Lake Fordyce (Fordyce), Lake Wohlford (Escondido), Mud Mountain (Stevans).)

Streams and dam locations, by State or country, are shown. Abbreviations, such as S.F., M.F., etc., preceding the name of a stream, indicate South Fork, Middle Fork, etc., of that stream.

Dams under construction are indicated by "u.c." In some cases, the construction of dams so designated may actually have been completed.

Proposed dams have been omitted, because published data are generally indefinite and in some cases are lacking entirely. Moreover, in the case of any dam merely in the study stage, there is the possibility of substitution of a different type of dam. In this respect, due to advanced stages of planning not known to the authors, some of the omissions may not be justified.

Dams which have failed have not been listed as such. Many, if not most, failures appear to have been due to inadequate spillway provision and not to type of dam or other design features.

Slopes of upstream and downstream faces are shown. The respective figures indicate the ratios which the horizontal component of a given slope bears to the vertical component.

Where different slopes occur on the same face, such are listed in sequence from bottom to top of dam, separated by commas, without details as to elevation at which the change occurs.

Under the caption "Slopes—Interior," applicable to earth cores and earth decks, are listed the slopes of the surfaces of separation between the earth and the major rock fill or fills. In the cases of central earth cores, the upstream earth slopes are stated first, then, following the semi-colon, the downstream slopes.

For central core-walls (other than earth) the sides of which are vertical or nearly so, the interior slopes are indicated with a single figure, "0.0."

Berms have been disregarded, except as affecting the average slopes.

Height of dam is aimed to be the height in feet of the cross-section, at the maximum section of dam, as defined in Section A. This has not always been possible, due to absence of cross-sections in the available descriptions; in such cases the reported height was used.

Type of dam and form of construction are indicated by means of symbols. Capital letters are used for the materials constituting the impervious parts or diaphragms, while small letters are used for other materials, as follows:

A = Asphaltic concrete
C = Portland cement concrete
E = Rolled earth
M = Masonry set in p.c. mortar
S = Steel
W = Wood

c = porous p.c. concrete
m = rubble (dry) masonry
r = rock fill

The sequence of the symbols, from left to right, conforms to the sequence of the materials of the dam from upstream to downstream. Thus, by way of example (with the numbers referring to the cross-sections in Fig. 1):

Ar—indicates an asphaltic concrete deck on rock fill (1).
Cmr—indicates a portland cement concrete deck, with rubble masonry, on rock fill (1).
rEr—indicates a central earth-core type of rock-fill dam (4).
Er—indicates an earth-deck type of rock-fill dam (2 and 3).

For p.c. concrete decks, no differentiation is made as between decks of various forms, such as the laminated, sliding or panel forms of construction.

In the cases of dams of the earth-deck type, differentiation between the relatively thin, earth-decks and those which are relatively thick seemed appropriate. Both being indicated by the symbol "Er," such differentiation is effected by the addition of an asterisk where the dam is of the relatively thin, earth-deck type; thus—Er*

Sources of Information

The information contained in the tabulations has been obtained from a great number of engineering periodicals of the United States and foreign countries, engineering hand-books, descriptive publications by Federal agencies, Transactions of International Congresses on Large Dams, etc., etc.

Particularly as to rock-fill dams in foreign countries, the "Statistical Review of Dam Construction," by Robert A. Sutherland, M. ASCE, (Proceedings, ASCE, Vol. 79, Separate No. 355, Nov. 1953) was helpful, though it did not furnish details permitting checking of type. Thus it was not feasible in every instance to determine whether a given dam, designated therein as "rock-fill" or "rock fill-earth fill" dam, is actually a rock-fill dam according to the definition proposed by the authors.

Likewise of great help was the recently published "Register of Dams in the United States," compiled by T. W. Mermel, M. ASCE, for the United States Committee on Large Dams. Taking advantage of its listing of the owners of dams, inquiries for details were addressed by the authors to such owners. On the whole, where inquiries reached them, the cooperation of those owners was excellent; the authors gratefully acknowledge their assistance.

In some instances, as disclosed by data supplied by the owners, designations for dams listed in the Register were found to be incorrect. Corrections in this respect have been incorporated in the pertinent tabulation.

Statement of the sources of information has been omitted, for otherwise the tabulations would have become much longer. However, the authors will be glad, upon request by interested readers, to supply for individual dams the references which were utilized.

Acknowledgments.—For corrections and additional data, the authors wish to express their gratitude to the following engineers who responded to the appeal for help contained in the original paper:

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TABLE 1.—ROCKFILL DAMS

Name of dam	Type and height, in feet	Name of river (creek where stated)	Location and year of completion	Slopes		
				Up-stream face	Interior	Down-stream face
ALPINE	Mrm 45	N.F., Stanislaus	Calif. 1906	0.0	-	0.28
ASHTON	Er 60	Snake	Idaho 1917	2.5	0.75	1.25
BARRE FALLS	Er 62	Ware	Mass. u.c.	2.25	1.3	2.00
BEAR CREEK	Er* 223	E.F., Tuckasegee	N. C. 1954	2.2	1.3	1.30
BEAVER CREEK	Sr 92	Beaver Creek	Colo. ?	1.7	-	1.2
BEAVER PARK	Cmr 87	Beaver Creek	Colo. 1914	0.5	-	1.5,0.5
BELDEN	rEr 150	N.F., Feather	Calif. u.c.	2.5	0.5;0.1	2.0
BONITA	Cmr 102	Rio Bonito	N. Mex. 1931	1.17	-	1.4
BOWMAN (North)	Cmr 175	Canyon Creek	Calif. 1927	0.75,0.50	-	1.4
BROWNLEE	Er* 297	Snake	Idaho 1958	3.0	1.37	1.4
BUCKHORN	rEr 162	M.F., Kentucky	Kentucky u.c.	2.5	9.33;0.33	2.5
BUCKS CREEK	Cmr 122	Bucks Creek	Calif. 1928	1.4	-	1.5
BULL CORRAL	Wmr 240	Huerfano Creek	Colo. 1911	1.0	-	1.25
CAGLES MILL	rEr 148	Mill Creek	Indiana 1953	2.0	0.5;0.5	2.00
CALISPEL	Cmr 82	N.F., Calispel Cr.	Wash. 1922	0.5	-	1.25
CEDAR CLIFF	Er* 165	E.F., Tuckasegee	N. C. 1952	2.2,1.3	1.3	1.3
CHATWORTH PARK	Cmrm 41	Mormon Creek	Calif. 1896	0.57	-	0.87
CHERRY VALLEY	rEr 330	Cherry	Calif. 1956	2.0	0.75;0.75	2.0
CHILHOWE	Er* 91	Little Tennessee	Tenn. 1957	2.2	1.3	1.3
CHRISTIAN VALLEY	Mr 33	S.F., Dry Cr.	Calif. 1916	0.5	-	1.25
CLEAR LAKE	Er 39	Lost	Calif. 1910	3.0	1.0	1.5
COGSWELL	WCmr 255	W.F., San Gabriel	Calif. 1935	1.35,1.3, 1.25	-	1.5
COUGAR	rEr 445	S.F., McKenzie	Oregon u.c.	2.0	0.5;0.0	1.75
COURTRIGHT	Cmr 310	Helms Creek	Calif. 1958	1.3,1.2, 1.1,1.0	-	1.4
CRANE VALLEY	rCr 145	N.F., San Joaquin	Calif. 1910	2.0	0.0	1.3
CUCHARAS	Wcmr 125	Cucharas	Colo. 1911	1.0	-	1.5
DALLAS	Er 295	Columbia	Ore.-Wash. 1957	2.3, 2.0	1.75	1.5
DAVIS	rEr 138	Colorado	Ariz.-Nev. 1950	8.0, 3.0	1.0; -0.25	3.0,7.5, 2.5
DIX RIVER	Cmr 275	Dix	Kentucky 1925	1.2,1.0	-	1.4,1.0
DREW	Wmr 65	Drew Creek	Oregon 1912	0.5	-	1.5
EAGLE GORGE	rEr 230	Green	Wash. 1957	2.15	?	1.3
EAST CANYON	rASAr 93	East Canyon Creek	Utah 1899	1.25	0.0	2.1
EAST FORK	Er* 135	E.F., Tuckasegee	N. C. 1955	2.2	1.3	1.3
FISH LAKE	Er 50	Fish Lake	Oregon 1956	3.0	1.25	1.25
FOURMILE LAKE	Cmr 25	Fourmile Creek	Oregon 1922	0.5	-	1.25
GLASIER LAKE	Cmr 52	Rock Creek	Montana 1937	1.0	-	1.5
GOOSE NECK	Scmr 210	S.F., South Platte	Colo. 1900	0.5	-	1.3
HELLS CANYON	Er* 320	Snake	Idaho 1958	2.5	1.37	1.4
HILLSIDE	Wmr 80	S.F., Bishop Cr.	Calif. 1904	0.75	-	1.25
HORSESHOE	rEr 194	Verde	Arizona 1945	2.0	0.4,0.3; 0.25	2.0
INLAND	Er 195	Warrior	Alabama 1938	3.0	1.3	2.0,1.3
LAKE FORDYCE	Cmr 130	Fordyce Creek	Calif. 1926	1.0	-	1.3

IN THE UNITED STATES

Name of dam	Type and height, in feet	Name of river (creek where stated)	Location and year of completion	Slopes		
				Up-stream face	Interior	Down-stream face
LAKE WOHLFORD	Wcmr 76	San Elijo Creek	Calif. 1895	0.5	-	1.25,1.0
LEMOLO # 1	Cr 120	North Umpqua	Oregon 1954	1.3	-	1.3
LEROY ANDERSON	rEr 260	Coyote Creek	Calif. 1950	2.5	0.75;0.75	2.5
LEWIS SMITH	Er* 290	Sipsey Fork of Warrior	Alabama u.c.	2.2	1.0	1.3
LITTLE HELL CREEK	rEr 33	Little Hell Creek	Colo. 1952	3.0	1.0;1.0	2.0
LOWER BEAR RIV. # 1	Cmr 233	Bear	Calif. 1952	1.3	-	1.4
LOWER BEAR RIV. # 2	Cmr 125	Bear	Calif. 1952	1.0	-	1.4
MEADOW LAKE	Cmrm 73	N.F., Mokelumne	Calif. 1903	0.75,0.5	-	1.0,0.5
MINIDOKA	Er 80	Snake	Idaho 1906	3.0	1.0	1.25
MONTGOMERY	Ar 113	M.F., South Platte	Colo. 1957	1.7	-	1.4
MORENA	CMmr 150	Cottonwood Creek	Calif. 1912	0.9,0.5	-	1.5
MUD MOUNTAIN	rEr 360	White	Wash. 1948	2.25,1.75	0.33; 0.33	2.25, 1.75
NANTAHALA	Er* 250	Nantahala	N. C. 1942	2.5,1.4	1.37	1.4
ONEONTA	Er 200	?	Alabama ?	3.0	1.0	2.0
OXBOW	Er* 170	Snake	Idaho u.c.	2.5	1.37	1.4
PARADISE	rEr 140	Little Butte	Calif. 1956	3.0	1.0;1.0	2.0
PELTON REGULATION	rEr 80	Deschutes	Oregon 1958	1.4	0.2;0.2	1.4
PENROSE-ROSEMONT	SMmr 100	East Beaver Cr.	Colo. 1932	0.5	-	1.4
PLEASANT VALLEY	Er 63	?	Utah 1927	3.0	0.75	1.5
QUEENS CREEK	Er* 78	Queens Creek	N. C. 1949	2.0	1.3	1.3
RELIEF	CMmr 140	Relief Creek	Calif. 1910	0.5	-	1.5
RHINEDOLLAR	Cmr 30	Leevining Creek	Calif. 1928	0.75	-	1.25
SABRINA LAKE	Wmr 55	M.F., Bishop Cr.	Calif. 1910	0.75	-	1.25
SADDLE BAG	Wmr 47	Leevining Creek	Calif. 1921	0.75	-	1.25
SALT SPRINGS	Cmr 328	N.F., Mokelumne	Calif. 1931	1.3	-	1.4
SAN GABRIEL # 1	Er* 375	San Gabriel	Calif. 1938	3.0,1.3	1.2	3.0
SAWMILL LAKE	Cmrm 55	Canyon Creek	Calif. 1910	1.0	-	1.0
SKAGUAY	Smr 70	Beaver Creek	Colo. 1901	0.58	-	1.2
SLY PARK	rEr 180	Sly Park Creek	Calif. 1955	2.5	1.0;0.8	2.0
SMITH	rECr 43	Trinchera Creek	Colo. 1916	1.7	0.0	1.3
STRAWBERRY	Cmr 150	S.F., Stanislaus	Calif. 1916	1.2,1.0	-	1.3
SWIFT	rCmr 165	Birch Creek	Montana 1914	1.5,1.2	-	1.33
TENNESSEE CREEK	Er* 185	E.F., Tuckasegee	N. C. 1955	2.2	1.3	1.3
THOMASTON	rEr 142	Naugatuck	Conn. u.c.	2.0	1.4;0.25	1.5
THREE LAKES	Cmr 30	Bucks Creek	Calif. 1896	1.25	-	1.5
TIOGA LAKE	Wmr 18	Leevining Creek	Calif. 1928	0.75	-	1.4
TRINITY	rEr 465	Trinity	Calif. u.c.	4.0,2.5	1.0; 0.75	3.0, 2.5
TUTTLE CREEK	rEr 157	Big Blue	Kansas u.c.	3.5	0.25;0.25	3.0
UPPER BEAR RIVER	Cmr 75	Bear	Calif. 1932	0.75,0.5	-	1.3,0.5
WATAUGA	rEr 328	Watauga	Tenn. 1949	2.0	0.85;0.85	2.0
WISHON	Cmr 270	N.F., Kings	Calif. 1958	1.3,1.2, 1.1,1.0	-	1.4
WOLF CREEK	Er* 185	Wolf Creek	N. C. 1955	2.2	1.3	1.3

TABLE 2.—ROCKFILL DAMS

Name of dam	Type and height, in feet	Name of river (creek where stated)	Location and year of completion	Slopes		
				Up-stream face	Interior	Down-stream face
AGOSTITLAN	rEr 92	Agostitlan Cr.	Mexico 1954	2.0	0.75;0.75	2.0
ALPE-CAVALLI	Cm 135	Loranco	Italy 1926	0.7	-	1.33
AMBUKLAO	rEr 420	Agno	Philippines 1955	2.0,1.75	0.25; 0.25	2.0,1.75
AURSJO	WAWcmr 123	Aura	Norway 1957	1.38,1.24, 1.1,1.0	-	1.38
BEAUFORT WEST	Cmr 185	Gamka	U. of S. Africa 1955	1.25	-	1.40
BERSIMIS	Er* 200	Bersimis	Canada 1957	2.5,2.0	1.38	1.38
BIA	Cmr 62	Lufira	Belgian Congo 1950	1.2	-	1.4
BOU-HANIFIA	CAr 175	Oued El Hammam	Algeria 1943	0.8;1.0	-	1.33
CARITAYA	CAcmr 125	Caritaya	Chile 1935	1.5	-	1.5
CHARCAS	Cmrm 49	?	Mexico 1933	1.25	-	1.4
CHIHUAHUA	rEr 151	Chuviscar	Mexico u.c.	2.0	0.5;0.5	2.0
COGOTI	Cr 245	Huatulame	Chile 1939	1.6	-	1.8
CODELAGO	MAMrm 64	?	Italy 1893	1.4,0.4	-	1.0
COOBY CREEK	Cmr 100	Cooby Creek	Australia 1941	1.2	-	1.5,1.0
CUARENTA	rEr 138	Lagos	Mexico 1949	2.25	1.0;1.0	2.25
DANXHO	rEr 103	Coscomate	Mexico 1949	2.5	1.0;1.0	2.5
DERBENDI KHAN	rEr 410	Diyala-Sirwan	Irac u.c.	2.25,1.75	0.3; 0.3	2.0,1.75
DESROCHES	Er* 200	Desroches	Canada 1957	2.5,2.0	1.38	1.38
DEVERO	AMmrm 74	?	Italy 1921	1.0,0.5	-	1.0
EILDON (New)	rEr 260	Goulburn	Australia 1956	2.5	0.6;0.75	2.5
EL TINTERO	rEr 184	Santa Maria	Mexico 1950	2.0	1.0;0.75	2.0
EL TUNAL	rEr 204	Tunal	Mexico u.c.	2.0	0.6;0.6	2.0
FURNAS	Er 394	Rio Grande	Brazil u.c.	2.0	0.5	1.80
GENKEL	Ar 140	Genkel	Germany 1952	2.25	-	1.75,1.5
GHRIB	CAr 213	Oued Cheliff	Algeria 1932	0.67 + 0 1.0	-	1.25
GLENBAWN	rEr 251	Hunter	Australia 1957	4.0,3.0	1.0; 1.0	5.0, 3.5,2.5
GOSCHENEN	rEr 510	Goeschenereuss	Switzerland 1960	3.0, 2.63, 2.0	0.15; 0.15	2.0, 1.4
GUADALUPE	Ecr 92	Guautitlan	Mexico 1947	2.0	1.0	3.0,1.3
GUADALUPE	rEr 131	Papigochic	Mexico u.c.	2.0	0.5;0.5	2.0
HANABANILLA	rEr 154	Hanabanilla Cr.	Cuba 1957	2.0	0.2;0.2	2.0
HARSPRANGET	rECEr 165	Stora Lule	Sweden 1951	1.5	0.0	1.5,1.0
HENNE	Ar 200	Henne	Germany 1954	2.25	-	1.75,1.5
HIGH ASWAN	rEr 361	Nile	Egypt u.c.	-	-	-
HIRFANLI	Er* 263	Kizil Irmak	Turkey 1958	2.4,1.3	1.3	1.3
HUICHAPAN	Cmr 174	Hondo	Mexico 1939	1.2	-	1.4
HUITZUCO	rEr 98	Huitzuco	Mexico u.c.	2.0	0.75;0.75	2.0
IRIL-EMDA	CAcr 246	Oued Agrioun	Algeria 1954	1.6	-	1.4
ISHIBUCHI	Cmr 172	Isawa	Japan 1953	1.4,1.3,	-	1.4
JYLHAMA	rEWEr 48	Oulujoki	Finland 1949	2.5	0.0	2.5
KAJAKAI	rEr 328	Helmand	Afghanistan 1954	2.5	0.4;0.0	2.0
KALKFONTEIN	Cmr 113	Riet	U. of S. Africa 1938	1.20	-	1.40
KARACHUNOVSKIA	Cmr 73	?	Russia ?	1.25,1.1, 1.0	-	1.5
KENNY	Er* 324	Nechako	Canada 1952	2.5	1.37	1.75
LA CONCEPCION	rEr 125	Tepotzotlan	Mexico 1949	2.5,2.0	0.75; 0.75	2.5,2.0
LA ESPERANZA	Cmr 88	Tulancingo	Mexico 1943	1.4	-	1.4
LAGARTIJO	Sr 79	Lagartijo	Venezuela 1956	1.5	-	1.5
LA JOIE	Wr 285	S. F., Bridge	Canada 1955	1.2,1.2 1.1,1.0	-	1.5
LASELE	rEr 75	Angerman	Sweden 1957	2.0	0.3;0.3	2.0
LA SOLEDAD	rEr 151	Santa Ana	Mexico 1955	2.0	0.5;0.5	2.0
LAUGHING JACK	Er* 40	Powers Cr.	Tasmania 1957	1.33	1.33	1.33
LA VEGA	rEr 59	Ameca	Mexico 1956	2.0	0.75;0.75	2.0
LE MARINEL	Er* 230	Lualaba	Belgian Congo 1955	2.7, 1.45	1.0	1.3

IN OTHER COUNTRIES

Name of dam	Type and height, in feet	Name of river (creek where stated)	Location and year of completion	Slopes		
				Up-stream face	Interior	Down-stream face
LEOBARDO REYNOSO	rEr 131	Lazos	Mexico 1949	2.25	1.0;0.75	2.0
LIGGA	rEr 120	Stora Lule	Sweden 1954	1.6	0.5;0.5	1.8
LOKVARKA	rEr 160	Lokvarka Cr.	Yugoslavia 1955	1.85	0.8;0.0	2.0
MADERO	Cr 163	?	Mexico 1938	1.20	-	1.4
MAKIO	rEr 266	Kiso	Japan u.c.	3.0,2.0	0.2; 0.2	3.0,2.0
MALPASO	CMmr 255	Mantaro	Peru 1936	0.50	-	1.5,1.33
MANUHERIKIA	CMmc 110	Manuherikia	New Zealand 1935	1.5,1.25	-	1.5
MARMORERA	rEr 292		Switzerland 1954	3.0, 2.25	-	2.0
MARY KATHLEEN	Cmr 75	Corella	Australia 1956	1.3	-	1.3
MELTON	rECr 116	?	Australia 1916	2.0	0.1;0.0	2.0
MESSAURE	rEr 330	Lule	Sweden u.c.	2.0, 1.75	0.17; 0.17	1.65
MIDSKOGFORSEN	rECr 92	?	Sweden 1944	2.0	0.0	1.5
MIBORO	Er* 415	Sho	Japan u.c.	2.5	0.85	1.75
MIGUEL ALEMAN	rEr 151	Tlaltenango	Mexico 1949	2.25	1.0;0.75	2.0
MOSSEL RIVER	Cmr 81	Mossel	U. of S. Africa 1948	1.40	-	1.40
MT. ISA	Cmr 87	?	Australia 1957	1.3	-	1.3
NOZORI	Cmr 145	Nozori	Japan 1956	1.3	-	1.4
OHRIGSTAD	Cmr 170	Ohrigstad	U. of S. Africa 1955	1.40	-	1.40
OSBU	WAWcmr ?	Lille	Norway 1958	1.37,1.25, 1.11,1.0	-	1.38
OUED-KEBIR	rCmr 115	Oued-Kebir	Tunisia 1925	1.5,1.0	0.0	1.5,1.0
PARADELA	Cmr 367	Cavado	Portugal 1956	1.3	-	1.3
PARRAL	rEr 78	Parral	Mexico 1951	2.0	1.6;1.6	2.0
PEGO DO ALTAR	SWAmr 213	Sado	Portugal 1949	1.25	-	1.4
PENA BLANCA	rEr 98	Tepozan	Mexico 1958	2.0	0.75;0.75	2.0
PENA DEL AGUILA	rEr 82	Sauceda	Mexico 1954	2.0	1.0;1.0	2.0
PERSEVERANCE	Er* 175	Perseverance Cr.	Australia u.c.	2.5,2.0, 1.4	1.3	1.3
PERUCA	rEr 200	Cetina	Yugoslavia 1958	1.45	0.1;0.0	1.5
PINZANES	Cmr 220	Tingambato	Mexico 1956	1.2	-	1.3
PRESIDENTE ALEMAN	rEr 249	Tonto	Mexico 1955	2.5	0.75;0.75	1.75
PRINS	Cmr 113	Prins	U. of S. Africa 1913	1.25	-	1.4
QUIROZ	rEr 164	Chipillico	Peru 1958	2.5	?	2.5
QUOICH	Cmr 110	Loch Quoich	Scotland 1956	1.3	-	1.4
ROCKY VALLEY	rEr 100	East Kiewa	Australia 1957	3.5,2.0	0.8; 0.8	2.0
ROTGULDEN	Ar* 60	Rotgulden Cr.	Austria 1958	1.25	1.0	2.0,1.25
RUST DER WINTER	Cmr 110	Elands	U. of S. Africa 1934	1.20	-	1.40
SAN ILDEFONSO	Cmr 203	Prieto	Mexico 1942	1.4	-	1.4
SANTO TOMAS	rEr 171	Ixtapandel Oro	Mexico 1956	1.75	?	1.41
SILLRE	Er* 26	?	Sweden 1933	2.0	1.6	1.5
SILVERWOOD	rECr 80	Rosenthal Cr.	Australia 1928	1.5	1.2;0.0	2.0
SLOTMOBERGET	rEr 110	Bard	Norway u.c.	1.65	0.85; 0.0	1.45
SONGA #1	rEr 155	Songa	Norway u.c.	1.7	0.8;0.1	1.5
SONGA #2	rEr 135	Songa	Norway u.c.	1.7	0.8;0.1	1.6
TACOTAN	rEr 226	Ayuquila	Mexico 1958	3.0,2.25	0.8;	3.0,2.0
TAXHIMAY	Cmr 137	Tepeji	Mexico 1934	0.75	-	1.4
TORPSHAMMAR	rECr 73	?	Sweden 1943	2.0,1.5	0.0	1.5
TRANGSLET	Er* 400	East Dal	Sweden u.c.	2.5	1.3	1.3
TUSTERVATN	rEr 60	Rossaen	Norway 1957	2.0	0.1;0.1	2.0
UPPER YARRA	rEr 293	Yarra	Australia 1957	3.5,3.0, 2.5,2.0	0.0; 1.0,0.3	3.5,3.0, 2.5,2.0
UREPETIRO	rEr 92	Tlazazalca	Mexico u.c.	2.0	0.5;0.5	2.0
VALSEQUILLO	rEr 269	Atoyac	Mexico 1946			
VRLA II	CAmr 82	Vrla	Yugoslavia 1955	1.4	-	1.5
WAYATINAH-B	Er* 65	Derwent	Tasmania 1957	1.5	1.33	1.33
ZUCUIRAN	Cmr 151	Zucuiran	Mexico 1957	1.4,1.3	-	1.4

DISCUSSION

F. L. LAWTON,[4] F. ASCE.—The authors are to be congratulated on an excellent analysis of the basic features and types of, and statistics on, rockfill dams.

The paper notes that "Constructed as expedients during pioneer days in the West, several of those early, though smaller, rock-fill dams remain as functioning monuments to ingenuity and engineering enterprise." This is true. However, it should not be deduced therefrom that these early dams were born "out of the blue". It is entirely probable their line of descent is from the rock-fill timber-crib dams which the early comers to eastern North America built to assist in logging operations. Some of these reached the notable heights of 70 to 100 feet.

The authors have provided an excellent classification of rock-fill and earth-fill dams but it is questionable if the use of the terminology "deck type" is sound. The accepted meaning of the word "deck" is a platform or covering extending horizontally across a vessel. While there are variants in meaning, depending on the country, the concept as applied to rock-fill dams is wrong. It is suggested "membrane" is more correctly applicable to those rock-fill dams which the authors have classified as deck type.

It is particularly interesting to note the authors have suggested that ". . . elimination of an excess of fines remaining in the run-of-quarry output might be accomplished by the use of (dump) trucks with slotted bottoms, thus permitting fines to be shaken out of the truck loads en route from quarry to dumping site." Certainly the selective dumping of quarry-run rock is relatively ineffective and decidedly uneconomic as a means of eliminating deposition of fines, although ample properly-directed high-pressure sluicing can go a long way towards overcoming settlement trouble arising from excess fines.

G. ROBERT KOCH,[5] A.M. ASCE.--The authors are to be congratulated for their informative paper which constitutes a very useful source of reference for engineers seeking statistical data on rockfill dams. The list of "Rockfill Dams in other Countries" presented by the authors is most comprehensive. However, it is difficult to make a complete list and the writer has submitted some additional data which includes some major dams.

The organization with which the writer is associated is presently engaged in the design of the 370 ft high, 10,500,000 cu yd central-core Akosombo rockfill dam in connection with the Volta River Hydroelectric Project for the Government of Ghana. This is the dam listed by the authors in their paper as Volta Dam. In the course of research on other major rockfill dams, certain supplementary statistics to the authors' list of "Rockfill Dams in other Countries" were gathered. These are presented herewith:

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Name of Dam	Total Volume of Embankment, in Cubic Yards
Bou-Hanifia	1.0
Furnas	12.0
Ghrib	0.9
Goschenen	11.7
High Aswan	55
La Joie	3.7
Marorera	3.5
Messaure	14.0
Pego do Altar (Salazar)	0.5
Pinzanes	0.5
Valsequillo	1.0

It is of interest to note that present planning is to create two of the world's largest man made reservoirs with rockfill dams: 120,000,000 acre-feet at Akosombo, and 105,000,000 acre-feet at High Aswan. Akosombo Dam is not included in the list because it is not yet under construction.

Although the additional dams supplied by the writer qualify as rockfill dams on the basis of the authors' proposed definition (that a rockfill dam is one of which at least half the material in the maximum cross section is comprised of loose rock placed by dumping), the writer feels that most engineers would prefer the definition whereby a rockfill dam is one which relies on dumped rock as a major structural element.

The writer gratefully acknowledges the cooperation of the engineers from foreign countries who furnished him some of the data presented to the authors. The writer also wishes to thank his colleagues Lee Worth and J. B. Cooke for supplying him with drawings and information pertaining to a number of the dams.

JOHN B. SNETHLAGE,[6] F. ASCE, F. W. SCHEIDENHELM,[7] and ARTHUR N. VANDERLIP,[8] F. ASCE.—The writers are pleased with the response to their paper, particularly from abroad. They had expected, however, some comment and suggestions regarding their definition of what constitutes a rockfill dam, as distinguished from dams in which earth is used in conjunction with rockfill. Thus, until further developed by others, the writers assume that their definition has been acceptable to the engineering profession, to wit:

> A rockfill dam is one of which at least half the material in the maximum cross-section is comprised of loose rock placed by dumping.

Mr. Lawton objects to the use of the name "deck" for the impervious cover of a rockfill and suggests that it be named "membrane." The word "deck" was selected from a series of names, all applicable (including membrane), as used at various times by others. It seemed that the word "deck" was the most suitable and had, moreover, the appeal of brevity; they would like to retain it.

However, should it appear in future dam literature that engineers, who actually work with this type of dam, find a better name than "deck," the writers will be glad to agree to substitution.

As explained in the original paper, the writers sought to bring, with their definition of rockfill dams, some order into the confusing nomenclature applicable to dams, which, of necessity, must contain, besides rockfill, a second material to form the water barrier.

For example, a dam consisting of 65% earth and 35% rockfill should not be termed a rockfill dam, for no one would deny that the 35% portion is not "a major structural element" of that dam on which reliance is placed. Mr. Koch's proposal is, therefore, surprising, even though he claims to interpret the preferences of "most engineers."

The writers agree with Mr. Koch that "it is difficult to make a complete list" and thank him for his additions. However, attention must be called to a few apparent errors.

Bon-Hanifia and Ghrib Dams.—These two dams are not rockfill dams, neither by Mr. Koch's nor by the writers' definition.[9] The deck of both dams, constructed on a steeply inclined wall of masonry set in cement mortar, consists of two layers of porous concrete separated by about 4 in. of asphaltic concrete.[10,11] Accordingly, the symbol for each of these two masonry dams should be cAcMm, rather than CAr.

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[6] Cons. Engr., East Chatham, N. Y.

[7] Cons. Engr., New York, N. Y. (Deceased).

[8] Cons. Engr., New York, N. Y.

[9] Engineering News-Record, Vol. 119, 1937, p. 891 and Vol. 120, 1938, p. 751.

[10] "Rockfill Dams: Montgomery Dam With Asphaltic Concrete Decks," by F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip, Transactions, ASCE, Vol. 125, Part II, 1960.

[11] Bitumen, German Asphalt Assoc., January, 1951.

Furnas Dam.--A paper[12] by James W. Libby shows a cross section of the Furnas Dam that indicates the height of the dam to be 102 m or about 335 ft.

Goschenen Dam.--From an available proposed cross section of this dam,[13] it is seen that the height of the main body of the dam, resting on gravel and boulders to be left in place, is about 300 ft; the central earth core, going down to rock, is shown with a height of about 395 ft. Perhaps, however, the design has been changed since the initial announcement.

LaJoie Dam.—The steepness of the upstream face, as listed by Mr. Koch, indicates the presence of at least a rubble masonry wall. The symbol could, therefore, be Wmr, rather than Wr.

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[12] "Furnas Hydroelectric Project," by James W. Libby, Proceedings, ASCE, Vol. 86, No. PO 2, April, 1960.

[13] Water Power, February, 1955.

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OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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